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PRESTRESSING CONTINUOUS COMPOSITE STEEL-CONCRETE BRIDGES

by

Frank W. Sarnes, Jr.

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A Thesis

Presented to the Graduate Committee

of Lehigh University

in Candidacy for the Degree of

Master of Science

in

Civil Engineering

Lehigh University

1975

CERTIFICATE OF APPROVAL

This thesis is accepted and approved in partial fulfillment of the requirements of the degree of Master of Science.

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Date

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ABSTRACT

A pilot investigation was undertaken to determine the feasibility of prestressing the slab in the negative moment region of continuous composite bridge beams. Two simply supported composite steel-concrete beams with prestressed slabs were used to simulate the prestressed negative moment region of a two-span continuous composite beam. In one beam the prestress force was applied prior to making the shear connection while in the other the prestress force was applied after the shear connection was made. The beams were tested under static loading and the results of those tests are presented herein.

An analysis of these results shows that slab prestressing in the negative moment region can be used to eliminate problems which previously frustrated the use of continuous composite beams, namely, slab cracking, fatigue of the tension flange of the steel beam due to the presence of welded connectors, and inefficient use of the slab in the negative moment region.

This analysis of the test results also indicates that existing specifications and design procedures can be

used to design continuous composite bridges if some minor modifications are made. When the slab is adequately prestressed, the transformed area method should be used for computing section properties and stresses in the negative moment region. The resulting increase of the slab force in this region must be considered in the design of the shear connection for both fatigue and ultimate strength requirements.

Design examples are presented to illustrate the problems encountered in the design of continuous composite bridge beams and to provide the basis for an economic comparison of continuous composite bridges in three categories: nonprestressed, prestressed before shear connection and prestressed after shear connection.

A study of the test results, design examples and the economic comparison leads to the conclusion that continuous composite bridges with slab prestressing in the negative moment region are a desirable economic alternative to conventional nonprestressed continuous composite bridges. They have a competitive construction cost, exhibit improved structural behavior and have a potential for reduced maintenance and repair costs over the service life of the structure. In addition, greater benefits can be realized when the prestressing force is applied after the shear connection has been made and a level of prestress is used which is higher than that required to prevent slab cracking under working load.

Finally, further research is required to develop more comprehensive design recommendations. This research should investigate the parameters governing the use of concentrated connector groups, the optimum level of prestress and the fatigue behavior of all components of prestressed continuous composite bridges.

1. INTRODUCTION

Two developments which have recently appeared in the design of bridge superstructures are the frequent use of continuity and the increasing use of composite action between the concrete deck and the steel beams. Both of these developments have resulted from the necessity for a more efficient and economical use of materials.

Until now, attempts to combine both continuity and composite construction have been frustrated by the lack of suitable methods of analysis and specific design recommendations and specifications. Previous designs could lead to premature fatigue failure of shear connectors near the dead load points of inflection and undesirable cracking of the deck in the negative moment (slab in tension) regions of continuous composite bridge beams.

Recent research at Lehigh University has concentrated on providing suitable analytical and design recommendations which would result in satisfactory designs based on static and fatigue criteria and minimize slab cracking.[1,2,3,4,5] Although full-scale tests indicate that these recommendations will result in improved structural behavior of the superstructure, several problems still remain.

Some transverse cracking of the concrete deck in the negative moment regions will always occur whether or not connectors are placed continuously throughout those regions. This cracking can lead to a more rapid deterioration of the concrete slab and reinforcing steel resulting in higher maintenance costs.

The effectiveness of the tension flange of the steel section will likely be reduced in negative moment regions by fatigue requirements. Current specifications limit the range of stress allowed due to live load plus impact when connectors are welded to the tension flange.[6,7] This problem becomes quite significant when shear connectors are placed continuously throughout the negative moment region. Finally, the full slab in negative moment regions is not as effective in resisting live loads as it is in the positive moment regions. Recent research has shown that a maximum of 20% of the slab remains effective under negative moment.[4,8]

At an early stage in the research it was realized that all three of these problems might be overcome by longitudinal prestressing of the negative moment regions. Transverse cracking of the slab could be eliminated over a certain range of live loads depending upon the level of prestress. Through the use of prestressing, the full slab could be made effective in resisting live loads. This has two beneficial effects. First, real continuity is achieved when connectors are placed continuously throughout the beam, resulting in more efficient use of materials. Secondly, the position of the neutral axis in the negative moment regions can be controlled by the designer thus making it possible to regulate the live load plus impact stress range in the tension flange of the steel beam and thus minimize the impact of fatigue design considerations.

With prestressing of the negative moment regions, however, additional questions are encountered which require further consideration before this procedure can be recommended for design. For instance, there are several ways to prestress the concrete slab depending upon the method and stage of construction. One construction method might be to cast the slab compositely with the steel beams in the usual way and to prestress after the slab has reached its desired strength. A second construction method might be to cast the slab non-compositely with the steel beams, prestress the slab, and then make the slab composite with the steel beams.

There are several ways the shear connection could be made after the slab is prestressed. For instance, holes could be left in the slab through which connectors could be welded to the steel beam and then grouted into the slab. The holes could either be made small enough to receive individual connectors or large enough to receive a concentrated group of connectors. The connector groups could then be spaced a certain distance apart.

Still a third construction method may be to use transverse precast slab elements in the negative moment regions which are prestressed in the longitudinal bridge direction either before or after these elements are made composite with the steel beams.

With this method, it is likely that the connectors would be grouped or concentrated into spaced out connector groups. Full width precast elements could be placed transversely across the beams between the connector groups. Shorter elements spanning across only two beams could be used adjacent to connector groups. The precast elements would be prestressed in the longitudinal direction of the bridge either before or after grouting of the connector groups.

The particular construction method used would probably depend on such factors as relative economics, structural advantages and ease of construction.

A pilot investigation was undertaken at Lehigh University to determine the feasibility of prestressing the slab in the negative moment region of continuous composite bridge beams and to evaluate two methods of providing the required shear connection; ie: equally spaced connector pairs and concentrated connector groups. The investigation was designed to provide analytical and experimental evaluation of the first two methods of construction mentioned above which do not involve precast slab elements.

Two composite test beams designated PSC-1S and PSC-2S were designed and constructed to simulate the negative moment region of a prismatic two-span continuous composite Tee-beam having the same span lengths and load positions as the beams previously described in Refs. 1, 2, 4 and 8. These two test beams were similar to and tested in the same manner as the test beams previously described in Refs. 9 and 10.

Both test beams used an equal number of stud connectors in each shear span, however, in one span the studs were placed in equally spaced pairs and in the other they were placed in three concentrated groups. Preliminary studies using computer programs developed and reported in Ref. 4 were used to predict the behavior of a beam with different stud arrangements on either side of the center support. These studies indicated that these arrangements would result in zero slip over the center support under symmetric loading, thus, each shear span would act independently and could be evaluated as such.

The major difference between the two test beams was in the procedure for making the prestressed slab composite with the steel beam. For beam PSC-1S, the concrete slab was cast and made composite with the steel beam in the usual way. After the concrete had reached the desired strength the slab was prestressed. Therefore, the transformed section resisted both the prestress force and the applied test loads. For beam PSC-2S, the connectors were isolated during casting and prestressing of the slab. The connectors were then grouted so that the transformed section would resist only the applied test loads.

The design and construction of the two test beams therefore made it possible to evaluate to some extent the feasibility of several construction techniques. The parameters considered were prestressing before shear connection, prestressing after shear connection, equally spaced shear connectors and grouped shear connectors.

This thesis investigates the effectiveness and practical applications of the use of prestressing to eliminate the problems mentioned at the beginning of this chapter which are presently encountered in the design and construction of continuous composite steel-concrete bridges.

The test results for beams PSC-1S and PSC-2S are presented herein. They provide the empirical data which is analyzed and discussed in comparison to theoretical predictions developed using current structural theory. This analysis and discussion is then used as a basis for the presentation and discussion of design considerations.

Finally, three different designs are presented for the same bridge. These designs use continuous composite bridge beams in three categories: nonprestressed, prestressed before shear connection, and prestressed after shear connection. These design examples are presented to illustrate problems encountered in the design of bridges in the three categories, to show how and when the existing specifications should be modified, and to provide a basis for an economic comparison of the three types of bridges.

2. TEST BEAMS, INSTRUMENTATION AND TESTING PROCEDURE

2.1 Description of Test Beams

Figure 1 describes the two test beams, PSC-1S and PSC-2S. As shown in Fig. 1, each test beam was constructed to simulate the negative moment region of the two-span continuous composite beam used in previous tests and shown in Fig. 1a.[1,8,9] The bending moment diagram due to the applied loads P for the two span beam is shown in Fig. 1b. The maximum positive and negative bending moments are shown as M_1 and M_2, respectively. For the two test beams shown in Fig. 1c, the test loads are shown as a shear V applied to the web at the location of the inflection points for the two span beam. In addition, the prestress force F is shown applied to the slab at the ends of the test beams. Figure 1d shows a typical cross-section for the test beams.

Each test beam consisted of a 60-in. wide by 6-in. thick reinforced concrete slab connected to a W21x62 (A36) rolled steel beam by 3/4 in. x 4 in. headed steel stud shear connectors. The location of the stud connectors is shown in Fig. 2 and described in detail later in this chapter (Art. 2.3). A prestress force was applied to the concrete slab of each beam by tensioning four 7/8 in. diameter regular grade STRESSTEEL bars centered in 1-3/8 in. diameter thin steel tubes as shown in Fig. 1d. The transverse slab reinforcement consisted of No. 5 reinforcing bars at 6-in. spacing top and bottom as described in previous reports.[1,8,9] Each beam had an overall length of 17'-0". This length included the 15'-0" negative moment region between the inflection points of the two span continuous beam and a 12 in. projection on each end as shown in Fig. 1c. The 15 ft. negative moment region also includes the 12'-8" region between the points of dead load contraflexure of the previously tested two span continuous composite beams.

2.2 Design Criteria

The design of the two test beams PSC-1S and PSC-2S was based on the design of the two-span continuous composite beam shown in Fig. 1a. Both the static and the fatigue design criteria were considered.[6,11] The resulting design, except for the slab prestress and the shear connection continuous through the negative moment region, was essentially the same as for those beams reported in Refs. 1, 8 and 9.

As in previous tests, with no slab prestress in the negative moment region of the two-span composite beam, the design of the shear connectors was controlled by fatigue requirements based on a working load P of 60 kips and a life of 2,000,000 cycles.[1] The working load level was determined by the allowable stress in the lower flange of the steel beam at the point of maximum positive moment.[1]

Assuming that the slab prestress would be used primarily to eliminate transverse slab cracking under at least working live loads, the prestress force F (Fig. 1a) was determined on the basis that at the previously obtained working load level P of 60 kips, the total stress in the slab over the interior support should be equal to zero. The resulting design was checked to ensure that the stresses in the steel beam and the remaining concrete slab due to P and F were equal to or less than the allowable stress. Assuming that for the steel beam $F_y = 36$ ksi and for the concrete slab $f'_c = 5$ ksi, the prestress force F was found to be 250 kips. It was decided that this prestress force would be applied to the continuous composite beam outside the inflection points and adjacent dead load points of contraflexure as shown in Fig. 1c.

Therefore, for test beam PSC-1S, which was made composite before prestressing, the prestress force of 250 kips was applied, as shown in Fig. 1c, prior to testing. For test beam PSC-2S which was made composite after prestressing it was decided to apply the same prestress force to the slab; ie: F = 250 kips. This was done to permit comparison of the effects of the prestress force in the two test beams.

2.3 Design Details and Fabrication

Fabrication details for test beams PSC-1S and PSC-2S are shown in Fig. 2. The rolled steel beams were cut from a 43-ft. length by a local fabricator. The bearing stiffeners and headed stud shear connectors shown in Fig. 2a and 2b were also welded to the beams by the fabricator. The excess length of the rolled shape was used for control test purposes and to test the stud welds. The welding procedure is described in Ref. 9 and the inspection procedure is outlined in Ref. 6.

Figure 2b shows the distribution of stud shear connectors for the two test beams. On one half of each beam the connectors were equally spaced in pairs along the top flange of the steel section. On the other half the connectors were arranged in concentrated groups. Each half of the beam contained the same number of stud connectors since the shear transfer requirements were the same for each half. This arrangement made it possible to study the effects of the two connector patterns and observe any differences. A typical cross section of the fabricated steel beam with connectors, stiffeners and loading pins is shown in Fig. 2c. The details for the stiffeners and loading pins were the same as in previous tests.(9)

2.4 Construction

Figure 3 shows the slab formwork with the transverse reinforcement and steel tubes for the STRESSTEEL bars in place for beam PSC-2S. Beam PSC-1S was identical with the exception that there was no formwork around the connectors. The connectors in beam PSC-2S were formed out so that the beam would remain noncomposite while the slab was cast and prestressed for the reasons previously mentioned (see end of Chapter 1). As shown in Fig. 3 extra top and bottom transverse reinforcement was placed at each end of the test beams to provide additional anchorage for the prestressing.

Two concrete mixes were used in the construction of these beams. The first mix was proportioned and transit mixed for a 28 day compressive strength of 3000 psi with a slump of 3-1/2 in. This concrete was used in lieu of the 5000 psi concrete used in the design so that a better comparison could be made with the results of previous tests in which 3000 psi concrete was also used for the slab of the test beams.[8,9] A second mix used as a grout for beam PSC-2S was proportioned for a 28 day compressive strength of 6000 psi with a slump of 1 in. and was mixed in a mechanical mixer in the laboratory.

Test beam PSC-1S was poured in one operation using the transit mixed 3000 psi concrete. Beam PSC-2S was also poured using the same concrete mix with the exception of the areas surrounding the stud connectors which were boxed out as shown in Fig. 3. After the slab of beam PSC-2S was prestressed, concrete made from the 6000 psi mix was placed in the voids around the connectors. This high strength, low slump mix was used to obtain a minimal amount of shrinkage.

Consolidation of the concrete on all of the pours was accomplished by internal vibration and the final finish was obtained by hand trowelling. The various pours took place on different days and control cylinders were made for each pour. Twelve cylinders were prepared from each batch of the transit mix concrete while eight cylinders were made from the high strength mix. In all cases half of the cylinders were made at the beginning of the pour and half at the end.

The concrete in the slabs was moist cured for seven days by covering the exposed surface with wet burlap and a polyethylene sheet. The forms were stripped at the end of the moist curing period and the slabs were allowed to cure under dry conditions until the beams were tested. Testing of each beam occurred approximately two months after the initial pouring of the slab. A number of control cylinders from each pour were cured in the same manner as the slab while the remainder were cured as prescribed by ASTM[12] (see Table 2).

2.5 Properties of the Test Beams

A test program was conducted to determine the mechanical properties of the materials used in the test beams. Properties of the structural steel were determined using tensile coupons cut from the excess lengths of the rolled beam (Art. 2.3). These coupons were tested according to the procedures outlined in Ref. 13. The mechanical properties of the rolled steel beam are shown in Table 1.

The concrete used for the slabs of the test beams was made of type 1 Portland cement, gravel crushed to a maximum size of 3/4 in. as coarse aggregate and natural bank sand as fine aggregate. Control cylinders were made for each batch of concrete (Art. 2.4). The results of the compression tests on these cylinders are shown in Table 2.

The physical dimensions of the test beams were obtained and the cross-sectional properties determined. The calculated properties are shown in Tables 3 and 4. In Table 3 the calculated properties for the rolled shapes are compared with the standard properties tabulated in the 1969 AISC Manual of Steel Construction.

2.6 Instrumentation

The instrumentation for test beams PSC-1S and PSC-2S was identical except that beam PSC-1S had strain gages mounted on the concrete slab while beam PSC-2S did not. Instrumentation details are shown in Figs. 4 and 5. A combination of electrical resistance strain gages, calibrated electrical slip gages and mechanical dial gages was used.

Figure 4 shows a detail view of one of the calibrated electrical slip gages. A 2-in. x 2-in. angle bolted to the underside of the concrete slab provides support for a small screw which bears against a metal strip which is secured to the top flange of the steel beam. Slip between the slab and the steel beam causes bending of the metal strip. Electrical resistance strain gages mounted on the metal strip record the bending strain in the extreme fibers of the strip. These strains are converted to slip (deflection at the screw tip) by means of calibration curves.

A mechanical dial gage, used to measure slip at the ends of the beam and to provide a check on the nearest electrical slip gage, is also shown in Fig. 4.

Figure 5 shows the locations of the nine instrumented cross sections used for both test beams. For both beams electrical slip gages were located at all nine locations. Ten strain gages were also mounted around the steel beam cross-section at each of the locations except 1, 6 and 9. The position of each of the ten gages on the cross-section of the beams is the same as in previous tests. They are shown in Fig. 4 of Ref. 9.

Test beam PSC-1S also had additional strain gages mounted on the concrete slab at locations 2, 3, 4, 5, 7, and 8. Locations 5 and 8 had a total of eight strain gages mounted on the concrete. At these locations, strain gages were mounted on the surface of the slab directly above and below each of the STRESSTEEL bars. At the remaining locations gages were mounted above and below the two interior STRESSTEEL bars only.

One of the interior STRESSTEEL bars was strain gaged for each beam in order that the prestressing force could be monitored throughout the test. The vertical deflection at each end of the test beams was measured by means of Ames 0.001 in. mechanical dial gages as shown in Fig. 5.

The recording equipment consisted of a B and F 100 channel automatic strain recorder and a Budd Datran unit to read the electrical resistance strain gages. A fifty power microscope was used to measure the width of the cracks which developed in the concrete slab near the end of the test.

2.7 Prestressing, Test Procedure and Loading

The four 7/8-in. diameter STRESSTEEL bars were prestressed to the desired level by means of the jacking system shown in Fig. 6. The system consisted of a fifty ton Simplex hydraulic jack with a jacking chair and a calibrated pull bar with coupler, pulling head and wedge. The pull bar was instrumented and calibrated to measure the applied tensioning force.

The prestressing operation was carried out in the following manner:

The pull bar was connected to the STRESSTEEL bar and the 50 ton jack with chair was placed over the pull bar and against the end plate. The pulling head and wedge were set in place and the jacking was started. When the load in the pull bar reached the required level, the nut on the STRESSTEEL bar was tightened until it was snug and bearing on the end plate. The jack was then released and the jacking force was completely transferred to the prestressing bar. The prestress force was applied in several stages to each bar in turn rather than all at once to any one bar to minimize the prestress losses due to the jacking sequence.

The loading procedure for the test beams was similar to that used in previous tests.[9] The center of the test beam was supported on a roller (this simulates the center support of the two-span beam) while vertical forces were applied to the web of the rolled steel beam as shown in Fig. 1c (this simulates the shear forces at the inflection points of the two-span beam). The test set-up is essentially the same as that used in previous tests and is shown in Figs. 9 and 10 of Ref. 9.

Each test was performed approximately two months after the slab was cast. Approximately 15 hours were required for testing which was carried out on 3 consecutive days. Load increments of ten kips were applied throughout each test until yielding of the steel beam was apparent. At this point small increments of deflection were used until the end of the test. The tests were continued until the unloading behavior of each beam was obtained.

3. TEST RESULTS AND ANALYSIS

3.1 Load-Deflection Behavior

Figure 7 shows the load-deflection behavior of test beams PSC-1S and PSC-2S. The curves show the relationship between the center reaction R and the deflection Δ of the center support relative to a line joining the load points as shown in the figure.

The deflection Δ of the test beam at the center support was computed as shown in the figure from the measured deflections Δ_1 and Δ_2 at the ends of the beam. This deflection is also assumed to represent the deflection of the inflection points relative to the center support of the two-span beam as shown in Fig. 7.

The predicted behavior of the test beams is shown by the dashed line. Since the properties of the two test beams were quite close (see Table 4), average values were used to determine one prediction curve for the two test beams.

For this prediction curve the full concrete slab was assumed to act compositely with the steel section until the first crack appeared. After initial cracking, it was assumed that only the prestressing steel would contribute to the composite behavior of the beam. For this reason, bilinear behavior was predicted for the elastic range and the transition occurs at the average cracking load of the two test beams.

Equations from the 1969 AISC Manual of Steel Construction were used to determine the deflections of the two span beam shown in Fig. 7 for the elastic portion of the theoretical curve. The predicted ultimate load was based on the plastic moment capacity of the composite steel section.

It can be observed from Fig. 7 that both test beams exhibited approximately the same behavior. This was expected since the section properties of both beams were almost the same. The slightly greater stiffness of beam PSC-1S can be seen at the higher load levels. The load-deflection curves for the two test beams are in good agreement with predictions and indicate that the full transformed section acts compositely approximately up to the cracking load when the slab is prestressed.

First yielding occurred in beam PSC-1S at a slightly higher level than in beam PSC-2S. This behavior was expected and is explained as follows. Beam PSC-1S was prestressed after the shear connection was made and, therefore, the entire transformed section was subjected to the prestressing force. This caused the extreme bottom fibers of the steel section to be in tension when testing began and thus these fibers reached their compressive yield stress at a higher load level than the extreme bottom fibers in beam PSC-2S which were not initially in tension. First cracking also occurred at different load levels in the two test beams and this will be discussed in detail in Art. 3.5.

In Fig. 7 it can be seen that both test beams exceeded the theoretical maximum load by about five percent which is probably due to strain-hardening. Due to local buckling, unloading took place immediately after the maximum load was reached; however, both beams were able to carry a large percentage of the maximum load for deformations considerably in excess of those at which local buckling occurred. The buckling was more severe in beam PSC-2S and probably explains why the unloading rate for beam PSC-2S was greater than that for PSC-1S. The local buckling of the two test beams will be discussed in Art. 3.6.

3.2 Bending Moment Distribution

Figure 8 shows the bending moment distribution for test beams PSC-1S and PSC-2S at two load levels. Load levels corresponding to center reactions of 60 and 150 kips have been used so that comparisons with previous test results are possible.[8,9]

The four lines are the theoretical predictions and the individual points show the test results. The theoretical values were computed by multiplying the applied shears by the measured moment arms and adding the moment due to the applied prestress force.

The effects of the prestress force was computed using the actual force in the instrumented interior STRESSTEEL bar. This force was assumed to be the force in all four bars and was multiplied by the eccentricity of the bars and then by four. The final product was a positive bending moment which was added to the negative bending moment computed using the applied shears to obtain the predictions shown in Fig. 8.

The different moment values at the point of application of the test shears resulted from the fact that at this point the negative moment due to the applied shears was always zero while the positive moment due to prestress increased due to the curvature of the beam under load. Test beams PSC-1S and PSC-2S differ at this point due to the fact that for PSC-2S the prestress force was applied to the slab only and thus the steel beam was not affected by this positive moment until loading began. On the other hand, PSC-1S was under a uniform positive moment of almost 800 in-kips before the loading began since the prestress force was applied after the shear connection was made.

The values shown as test results were computed using the strain readings from the 10 strain gages on the steel beam at each of six instrumented cross sections. These instrumented cross sections were previously discussed (Art. 2.6) and are labeled on the horizontal axis of Fig. 8. The strain readings were first linearized using a least squares fit. The linearized strains were multiplied by Young's modulus to convert them to stresses. The stresses were then integrated over the area of the steel beam to compute the horizontal force and bending moment in the steel beam at each section.

Due to the use of prestressing, the full transformed section was assumed to be effective in carrying the load and consequently had to be considered in the computations of the actual bending moment. Because of problems with the concrete mounted gages of PSC-1S and the fact that no gages were mounted on the slab of PSC-2S, basic composite theory was used to include the moment carried by the slab in the actual bending moment. By equilibrium the force in the concrete was assumed to be equal and opposite to the force in the steel beam. Since the neutral axis was very close to the slab-steel beam interface, and for simplicity, a triangular stress distribution was assumed for the concrete slab. Thus, the concrete force was applied at the one-third point in the depth of the slab and the resulting bending moment for the concrete was added to that computed for the steel beam resulting in the actual moments shown in Fig. 8. All of the above computations were made using the CDC 6400 computer at Lehigh University.

The actual bending moments computed from the test strains are in good agreement with the predictions for both test beams at the 60 kip load level. Although not shown on the figure, this close correlation between actual and predicted bending moment was observed in both test beams at load levels exceeding a center reaction of 110 kips. This is significant in that it supports the assumption that the full transformed section is carrying the live load. If this were not the case, the moment values computed using the actual strains in the test beams would have greatly exceeded the predicted moment values. (Since the force in the concrete slab was assumed to be equal in value to the horizontal force in the steel section, if the steel section was actually carrying most of the load the inclusion of the slab force and moment would have almost doubled the resulting test moments.)

At the 150 kip load level, however, this close correlation is only observed at the instrumental cross sections near the ends of the test beams. The values at the sections near the interior support are not in good agreement with the prediction. One value for PSC-2S is not shown because it is off of the scale.

This lack of correlation can be explained by two factors. First, due to cracking, the concrete slab over the interior support was less effective than that near the ends of the beams. Secondly, the electrical resistance strain gages from which the results were obtained lose much of their reliability at high strains. It was obvious that high strains were present since extensive yielding was observed over the interior support at this load level for both test beams.

The somewhat poorer results for PSC-2S in this region can be explained by the fact that the steel beam in PSC-2S yielded at a much lower load level (Art. 3.1). Therefore, higher strains were present and the strain gage readings were probably less reliable than those in PSC-1S.

3.3 Slab Force

Figure 9 shows the slab force distribution for PSC-1S and PSC-2S at the 60 and 150 kip load levels. The actual values of the slab force were computed in the same manner as the actual bending moments (Art. 3.2). The predicted values illustrated by the four curves were computed by applying the test shears to the beams and computing the force in the slab at each section using equilibrium and compatibility. The effect of the prestress force was then included. The full transformed section was again assumed to be effective and computations were made using the computer.

Since there were no stud connectors between cross sections 2 and 3 or between 4 and 5, the prediction follows a horizontal line between these points. This is also the case near the ends of the two test beams.

The predicted and actual values are in good agreement at the 60 kip load level. The actual values at all load levels indicated that some shear transfer does take place in the regions where no connectors were welded to the beam. This transfer is probably due to bond and friction forces between the concrete slab and the steel beam. The poorer correlation at the 150 kip load level resulted in part from causes discussed in Art. 3.2 concerning the poor correlation of bending moments at this load level.

Some problems were caused at all load levels by the fact that only one STRESSTEEL bar was instrumented. Thus the total prestress force used in the computations was not necessarily the total prestress force present in the test beams. This possible deviation in the prestress force had little effect on the bending moment as illustrated in the following and therefore was not discussed. For example, a 50 kip deviation in the total prestress force would require that the prediction curves in Fig. 8 be shifted in the vertical direction by approximately 0.067 inches, while this same 50 kip deviation would require that the prediction curves in Fig. 9 be shifted by 0.5 inches thus having a greater influence on the correlation of predicted and actual values.

Finally, it can be observed that if straight lines were drawn from the end values to the values at the center line, relatively good correlation would still result. This is significant since the straight line neglects the effect of connector grouping and indicates that grouped connectors can be designed in basically the same manner as uniformly spaced connectors. In other words, the number of connectors used is critical while the spacing is not if the connectors are reasonably distributed over the length of the shear span and individual connectors are spaced far enough apart to insure full development of their shear strengths. The exact parameters governing the locations of connector groups can only be determined by additional research.

3.4 Slip

Figures 10 and 11 show the slip distribution for the two test beams. The slip measurements were made using electrical slip gages as described in Art. 2.6. The least accurate of the slip gages was capable of recording a change in slip of 32 micro inches. In both figures, slip of the slab relative to the steel beam to the east is shown as positive.

Figure 10 shows the slip distribution obtained in both beams due to the applied test loads only (effect of prestress not included). As in previous figures, results at load levels corresponding to center reactions of 60 and 150 kips are shown. The magnitude of the slip in beam PSC-1S was generally greater than that in beam PSC-2S. This can probably be explained by the fact that in beam PSC-2S the concrete surrounding the connectors had a much higher compressive strength than the concrete surrounding the connectors in beam PSC-1S (see Table 2).

Unlike previous tests, no attempt was made to eliminate the bond between the steel beam and the concrete slab for either test beam prior to testing. The effect of this bond may be observed in Figs. 10 and 11 in the areas between cross sections 2 and 3 and between cross sections 4 and 5. Since there were no shear connectors in these two areas, the change in slip was probably due to this bond between the steel and the concrete as previously discussed (Art. 3.3).

The slip distribution in both beams followed similar patterns with the exception of the east half of beam PSC-1S at the 150 kip load level. However, at the prior load level of 140 kips the slip distribution in the east half of beam PSC-1S was similar to that in the east half of beam PSC-2S. While the test load was increased from 140 kips to 150 kips, a significant failure in the bond between the slab and the steel beam was observed and may account for the above noted variation in slip distribution.

It should also be noted in Figures 10a and 10b that although the magnitude of the slip in the west half of each beam was slightly greater than that at corresponding sections of the east half, the magnitudes were approximately the same. This indicates that the grouping of connectors in the west half of each beam did not greatly affect the slip behavior of that half of the test beam as compared to the half with equally spaced connectors.

Figure 11 shows the slip distribution in beam PSC-1S including the slip prior to testing caused by the prestress force and the losses which occurred during the time which elapsed between the prestressing operation and the application of the test loads. This figure indicates that initially the prestress force was carried by the connectors at the ends of the beam. With increasing time this force was redistributed across the length of the beam.

3.5 Cracking Behavior

Figures 12 and 13 show the crack patterns in the concrete slabs of test beams PSC-1S and PSC-2S. Figure 12 shows the crack patterns at a load level of 90 kips center reaction which is about $1\frac{1}{2}$ times working load level. Figure 13 shows the crack patterns at the maximum load level. In both figures, the rectangles shown within the slab of beam PSC-2S are the areas where the stud shear connectors were isolated from the main slab pour, and then grouted later using the high strength grout.

At the 90 kip load level, beam PSC-1S had one crack approximately 0.005 in. wide across the entire width of the slab directly over the center support while beam PSC-2S had 3 hairline cracks (approximately 0.01 in. wide) in the area where the grout was placed. The hairline cracks occurred in the grouted areas of beam PSC-2S due to the fact that the grout was placed after the slab was prestressed and consequently these sections of the slab were not actually prestressed.

It was expected that the slab of beam PSC-1S would crack before that of beam PSC-2S since the slab of beam PSC-2S carried the entire prestress force while the full transformed section carried this force in beam PSC-1S. For this reason, the slab of PSC-2S had a greater compressive stress at the start of the applied test loading.

It can be seen from Fig. 13 that even at the maximum load level there were very few cracks in the slab of either test beam. Beam PSC-1S contained three cracks across the width of the slab while beam PSC-2S had the equivalent of two cracks. The crack near the center support of beam PSC-2S was the first to form as was expected.

The cracks near the center support of each beam attained the greatest width. The average width of these cracks at the maximum load level was about 0.1 in. Most of the other cracks were hairline cracks with average widths generally less than 0.01 in. Both beams had short longitudinal hairline cracks at the east and west ends due to the high anchorage stresses induced by the prestressing bars.

The slab cracking in both beams is quite symmetrical about the center support. This is significant since each half of the two test beams had a different distribution of stud shear connectors. The symmetrical crack patterns indicate that the beam behavior was symmetrical about the center support.

The crack control benefits resulting from the use of prestressing in the two test beams can be observed when Figs. 12 and 13 are compared with the results of previous tests.[8,9] The beams used in these tests basically differed from beams PSC-1S and PSC-2S only in that they were not prestressed and did not have shear connectors placed continuously throughout the negative moment region. All of the non-prestressed test beams had numerous cracks across the full width of the slab at a load level of 60 kips. By comparison, the first crack in the prestressed test beams did not appear until the 90 kip load level. At this level only beam PSC-1S was cracked across the full width of the slab and this crack had a width of 0.005 in. or one-half of the maximum width for exterior members recommended by ACI in Ref. 14.

3.6 Failure Mode

Test beams PSC-1S and PSC-2S both failed in essentially the same manner as the non-prestressed beams which were previously tested.[8,9] For both test beams, failure occurred after local buckling of the compression (lower) flange of the steel beam adjacent to the center support. Local buckling of the flange was accompanied by web buckling in both beams. Web buckling in PSC-2S was quite pronounced as can be seen in Fig. 14. In both tests local buckling occurred shortly after maximum load was reached as shown in Fig. 7.

Figure 15 shows beam PSC-2S shortly after failure. This was typical of both test specimens. Local buckling of the lower flange and web of this beam can be seen near the center support.

3.7 Significance of Test Results

In Chapter 1, three problems were recognized which currently limit the use and effectiveness of continuous composite steel-concrete bridges. These problems were: 1) excessive transverse slab cracking under working loads, 2) lack of effectiveness of the concrete slab under negative moment, and 3) limited live load stress range permitted in tension flange when shear connectors are placed throughout the negative moment region.

The use of prestressing in the negative moment region was proposed as a solution to these problems. The test results presented in this chapter support this proposal by illustrating that prestressing the slab in the negative moment region is a feasible technique which can be used to eliminate the three problems listed above.

In both test beams, the prestress force prevented slab cracking at load levels well in excess of working load (Art. 3.5). Even after cracking occurred, the prestressing prevented the formation of the large number of cracks which were observed in previous tests.[8,9]

Since cracking was prevented, the concrete slab remained effective at load levels above working load. The full transformed section was therefore assumed to be effective in resisting the applied loads. This assumption was used throughout the analysis of the test results. The good correlation between the predicted and actual behavior of the test beams discussed in Arts. 3.1, 3.2 and 3.3 indicates that this assumption was valid.

This is a significant finding. It not only allows a more efficient use of the concrete slab in the negative moment region, but permits simple span composite beam design methods, currently used to design the positive moment region, to also be used in the design of the negative moment region of continuous composite bridges. This concept will be further discussed in Chapter 4.

Although the use of prestressing does not eliminate the fatigue requirement for a reduction of the live load stress range permitted in the tension flange of a steel beam with welded shear connectors, prestressing does reduce it to an insignificant problem. The application of a prestress force to the slab results in the full transformed section resisting the live load. This causes the neutral axis to shift to near the slab-steel beam interface and reduces the live load stress range to near zero. Furthermore, if this prestress force is applied after the shear connection has been made, it will reduce the magnitude of the maximum tensile stress to which the top flange is subjected. This will be further discussed in Chapter 4 and illustrated in Chapter 5.

The final significant test result is the effectiveness of the concentrated connector groups. Although further study is required to determine the limiting parameters, the symmetry of the test results for the two shear spans indicates that the connector groups transfered the horizontal shear forces as effectively as the equally spaced connectors. This finding makes prestressing before shear connection a more practical construction method than it would be if holes had to be left for individual stud connectors.

4. DESIGN CONSIDERATIONS

4.1 Introduction

Although further research is required before extensive and conclusive design recommendations can be made, the results of this pilot study provide sufficient knowledge of the behavior of prestressed continuous composite beams to permit their design using current composite bridge design criteria with some modifications. This concept and the required modifications will be discussed in this chapter and illustrated by the design examples contained in the Appendices.

4.2 Positive Moment Regions

As specified by AASHTO, the positive moment portion of continuous spans may be designed with composite sections as in simple spans.[6] This is also true for a continuous composite span with prestressing over the interior supports.

When the slab is prestressed after the shear connection has been made, however, the designer should consider the effect of the bending moment caused by the application of the prestress force. This moment can significantly reduce the maximum positive moment in this region and thus reduce the size of the composite section required.

4.3 Negative Moment Regions

When the prestressing force applied to the slab is sufficient to prevent cracking at or above working load (value of live load plus impact used for allowable stress design), the composite section in the negative moment region can be designed in the same manner as in the positive moment regions. Since the slab does not crack under working load, it can be considered fully effective. Using the transformed area method, the moment of inertia and extreme fiber stresses can be computed in the usual manner.

The limiting values for these stresses are in accordance with current practice and specifications. The stress at the bottom of the steel beam should be less than the allowable compressive stress for the given grade of steel and plate geometry. The stress at the top of the concrete slab should be less than the tensile cracking stress and can usually be taken as zero.

In addition to the above limitations on the extreme fiber stresses, the stress range under live load and impact in the top flange of the steel beam should not be permitted to exceed the allowable stress range for fatigue.[6,7] The stress range can easily be reduced by proportioning the composite section in the negative moment region to shift the neutral axis toward the slab-steel beam interface.

If the slab is permitted to crack under live load, the prestressing steel will act as a tie bar and enable the beam to continue to carry the load. Since the moment of inertia is reduced, deflection behavior of the structure will be changed. However, this study indicates that the change will not be significant at the working load level. Considering the high initial tensile stress in the prestressing steel, the beam will continue to behave in a satisfactory manner until either the prestressing steel yields or the compression flange exhibits local buckling. Corrosion and fatigue problems due to slab cracking should be considered, however, to prevent failure of the prestressing steel.

In addition to proportioning the composite section, the design of a prestressed continuous composite bridge requires a determination of the prestress force and shear connection necessary for satisfactory performance of the structure. Since these considerations are not directly covered by current specifications, they will be discussed in detail in the articles which follow.

4.4 Criteria for Level of Prestress Required

The level of prestress in the beam can vary from that required to prevent slab cracking prior to a load which is less than the working live and impact load to that required to prevent cracking up to a given overload. The optimum level will be that which results in the greatest material, maintenance and replacement cost saving per dollar spent to prestress the slab in the negative moment region of the beam. Although further research and field tests are required before this optimum level can be determined, the results of this study indicate two areas which should be considered.

As a mimimum, the level of prestressing force applied to the slab should be that to prevent cracking under working load. If the slab is permitted to crack below working load, these cracks will be exercised each time the bridge is stressed to the working load level.

Reflective cracking in the asphalt wearing surface will eventually result in deterioration of the pavement, as well as the reinforcement and the slab. Such a design would eliminate one of the major benefits gained through the use of prestressing, that is, reduced maintenance and replacement costs. The maximum level to which the slab can be prestressed is determined by the geometric properties of the composite section, the strength of the concrete and steel used, and the cost of the prestressing operation versus the resulting benefits.

4.5 Calculation of Prestressing Steel

Assuming that slab cracking is to be eliminated prior to working load, it is necessary that the stress in the top of the slab be nearly equal to zero or compressive under working live plus impact load. The use of zero tensile stress for crack control in concrete design is recommended by both AASHTO and ACI.[6,14]

When the slab is prestressed before the shear connection is made, the design of the prestressing steel is very simple. All design loads are applied to the nonprestressed structure and the stresses in the transformed section are computed assuming an uncracked slab in the negative moment region. Maximum tensile stresses in the concrete slab should occur over the interior supports.

The prestress force required can then be computed by multiplying the tensile stress by the effective cross-sectional area of the slab. The summation of the compressive stress introduced by the prestressing and the tensile stress caused by the design loading will result in a total stress of zero in the top of slab over the interior support under the design live plus impact loads.

When the prestressing force is applied after the slab has been made composite with the steel beam (that is, after the shear connection has been made), the determination of the required prestressing force is more complicated. In this case, the force is applied to the full transformed section and to the entire continuous member resulting in shears and bending moments in all spans. These stress resultants can be determined by applying the unknown prestress force (F) at an eccentricity (e) to the member. The stress resultants due to the application of the design loading to the nonprestressed member are also determined assuming an uncracked slab. The two sets of stress resultants are then superimposed and the required prestressing force determined.

If the prestressing is to be used only to prevent slab cracking, a prestress force (F) and eccentricity (e) are selected which result in zero stress in the concrete slab over the interior supports as well as allowable stresses in the compression flange and stress ranges acceptable for fatigue. Should it be desirable to reduce the stresses in the steel beam at given section, a combination of F and e is determined which will result in the desired stress condition at that section. These calculations are illustrated in the design example in Appendix C.

As in any bridge design, the stresses at critical sections must be calculated for the different loading conditions experienced by the bridge during the various stages of construction and service life. These stresses must be kept within allowable limits.

In accordance with good practice, the prestressing steel should be distributed uniformly across the effective width of the slab and prestressing losses must be considered. If the slab is prestressed before shear connection, the prestress force should be applied at mid-depth of the slab to prevent uplift. When the slab is prestressed after the shear connection has been made, the prestress force can be applied at any depth in the slab as long as sufficient concrete cover is provided.

Since the concrete slab in the negative moment region is to be prestressed, additional longitudinal reinforcement should not be required. However, should a long delay be required between the pouring of the slab and the post-tensioning operation, additional reinforcement should be provided to control the effects of temperature and shrinkage.

4.6 Design of Shear Connection

The shear connection should be designed in accordance with the specifications contained in Refs. 6 and 7 (Article 1.7.100). However, some minor modifications can be recommended to adapt them for use in the design of prestressed continuous composite bridges.

Except for one modification, shear connector fatigue requirements remain basically unchanged. However, when the slab in the negative moment region is prestressed, the statical moment, Q, in that region should be taken about the neutral axis of the composite section using the full transformed area of the slab. This is the same as currently used in the positive moment regions and should result in a conservative design since any slab cracking will reduce the shear connection force. Further research is required to better define the parameters involved in the design of shear connectors for fatigue in the negative moment region.

Shear connector ultimate strength requirements for the region between points of maximum positive moment and the end supports remain the same, however, they do not remain the same for the other regions of a continuous beam.

The formula for the minimum number of shear connectors, N_2, required between points of maximum positive moment and points of adjacent negative moment must be modified to include the increase in the ultimate slab force which results from the application of the prestressing force. The modified formula is as follows:

$$N_2 = \frac{P + P_3 + P_4}{\phi \ S_u}$$

All terms except P_4 are as previously defined in Refs. 6 and 7. At points of maximum negative moment, the force in the slab due to prestressing is taken as:

$$P_4 = A_s^p \ F_y^p$$

where: A_s^p = total area of the prestressing steel at the interior support within the effective flange width.

F_y^p = specified minimum yield point of the prestressing steel.

The shear connectors should be placed continuously throughout the negative moment region and consequently anchorage connectors are not required.

As in the tests reported herein, connectors may be arranged in concentrated groups. Until further study better defines the parameters, the spacing between the groups should be limited a maximum of 24 inches (the maximum pitch specified in Ref. 6) and the spacing between individual connectors should be limited to a minimum of 3 inches. This latter criteria is the same as that used in the test beams which performed satisfactorily.

5. DESIGN EXAMPLES

5.1 Introduction

Appendices A, B and C contain designs of continuous composite bridge beams in three categories: nonprestressed, prestressed before shear connection, and prestressed after shear connection. These designs are presented in the appendices and discussed in this chapter to provide design examples of continuous composite bridges with prestressed slabs in the negative moment region, to illustrate where the existing AASHTO specifications should be modified, and to provide the basis for an economic comparison of the three categories of continuous composite beams. In keeping with these objectives, many details are omitted which are normally part of a bridge design. Only items pertinent to this investigation are included. In addition, some of the beam cross sections are somewhat overdesigned. These designs have not been refined since that would add little to this study.

To obtain realistic values of shear and moment due to dead load (D.L.) and live load plus impact (L.L.+I.), a bridge analysis problem was selected from Ref. 16. The bridge to be analyzed and designed is shown in Figs. 16a and 16b.

The bridge was analyzed using the Clough displacement method and the CDC 6400 computer. The structure was divided into 20 ft. segments identical to the one shown in Fig. 16d and the stress resultants were computed for all fourteen analysis sections shown in Fig. 17a. For illustrative purposes, the interior Tee beam shown in Fig. 16c was chosen for use in the three design examples. The results of the analysis are tabulated in Table 5 and illustrated graphically in Figs. 17, 18 and 19.

The positive and negative moment regions in a continuous composite bridge are defined by the dead load points of contraflexure. Three design regions were chosen whose boundaries approximately correspond to these points. These regions are identified in Figs. 17c and 18a and are used extensively in the design examples to identify the portion of the beam being designed. The notation used in Appendices A, B and C is summarized in Appendix F.

5.2 Nonprestressed Continuous Composite Bridge Beam

The design example for a nonprestressed continuous composite bridge beam is contained in

Appendix A. All design calculations are in accordance with the AASHTO specifications.[6,7] The resulting composite beam sections for the three design regions are shown in Fig. 20.

5.3 Continuous Composite Bridge Prestressed Before Shear Connection

The design example for a continuous composite bridge beam with the slab in the negative moment region prestressed before shear connection is contained in Appendix B. All design calculations are in accordance with the AASHTO specifications.[6,7]

Two levels of prestress are considered. The first prevents slab cracking up to working load. The second prevents cracking up to one and a half times working load thus providing a factor of safety of 1.5 against slab cracking under working loads. The relative benefits of using the higher level of prestress will be discussed in Art. 5.5.

The composite beam sections for the positive moment regions (Regions 1 and 2) are identical to those obtained in Appendix A and are shown in Figs. 20a and 20b, respectively. For the negative moment region

(Region 3), the composite beam section is shown in Fig. 21a. The section shown is for the case when slab cracking is prevented up to working load. For the higher level of prestress, the section is the same except that 1 1/8 in. diameter Regular grade prestressing bars are used. In addition, for the section shown in Fig. 21a, the stress distribution for the different stages of loading is illustrated in Fig. 22a. It should be noted that only the concrete slab is affected by the prestress force.

5.4 Continuous Composite Bridge Prestressed After Shear Connection

The design example for a continuous composite bridge beam with the slab prestressed after shear connection is contained in Appendix C. All design calculations are in accordance with the AASHTO specifications.(6,7) The same two levels of prestress are considered which were considered in Appendix B.

Since the prestressing force is applied to the full transformed section and to the entire continuous structure, an analysis is required to determine the stress resultants due to the application of the prestress force. This analysis is contained in Appendix D and the resulting stress resultants are shown in Fig. 24. As recommended in Art. 4.5, these stress resultants are superimposed on those due to all other loads and the prestress force required to prevent cracking is determined. As shown in Appendix C, the application of this prestressing force substantially reduces the extreme fiber stresses in all three regions and permits lighter steel sections to be used.

The composite beam sections for the positive moment regions are identical to those shown in Figs. 20a and 20b, except that different beam sections were selected for most cases. When cracking is prevented up to working load, a W33x130 steel section was chosen for Region 1 while a W33x200 is still required for Region 2. For the higher level of prestress, W33x118 and W33x152 steel beams are adequate for Regions 1 and 2, respectively.

The negative moment section for the case when cracking is prevented up to working load is shown in Fig. 21b. The stress distribution for this section is illustrated in Fig. 22b. The effect of the prestress force on the full transformed section should be noted. For the higher prestress level, a similar negative moment section is used. The only changes are the removal of the coverplate from the steel beam and the use of five 1 3/8 in. diameter Regular grade prestressing bars.

5.5 Economic Considerations

The relative construction costs of the three beams designed in Appendices A, B and C are computed in Appendix E. All costs used are final in-place unit costs based on data listed in Ref. 15. These unit costs include the cost of the material, its delivery to the job site and construction labor. As in the designs, one-half of each beam is used for the cost comparison.

For both prestressed continuous composite beams and for both levels of prestress, the cost of the prestressing and the additional stud connectors required is more than offset by reduced costs for reinforcing steel, coverplates and steel beams. The net savings varies from $184 for the beam prestressed to the working load level before shear connection to $1619 for the beam prestressed to the higher level after shear connection. Thus, in addition to its improved structural behavior, a prestressed continuous composite bridge is competitive in construction cost with the nonprestressed continuous composite bridge.

In addition to providing a factor of safety against slab cracking, the higher level of prestress results in greater net savings for both prestressed beams. Designs with the slab prestressed after shear connection also yield higher savings than the designs where the slab was prestressed before the shear connection. As illustrated in Appendix E, the greatest net savings are realized when the slab is prestressed to the higher level after the shear connection has been made.

In addition to the construction cost savings, the prestressed bridges should require less maintenance and fewer slab replacements and overlays since slab deterioration will be slowed by crack prevention. The value of this maintenance cost saving can only be determined after a prestressed continuous composite bridge is actually constructed and has been in service for several years.

The construction and maintenance cost savings of a prestressed continuous composite bridge make it an economically desireable alternative to conventional continuous composite bridges and one worthy of further investigation.

Finally, consideration should be given to using levels of prestress in excess of that required to prevent slab cracking at one and a half times working load. The application of additional prestressing force will permit a more efficient use of the concrete slab in the negative moment region and will further reduce the size of the steel sections required for all regions of the beam when the prestress force is applied after the shear connection has been made. Further research is required to determine the optimum level of prestress.

5.6 Other Considerations

The composite beam sections developed for Region 3 in the design examples had a ratio of transformed concrete slab area to area of the steel section of less than one. This can be compared to the ratio used in the test beams of better than two and a half. It is obvious that greater benefits can be gained from prestressing the slab in the negative moment region if the slab carries a greater portion of the load. This ratio can be somewhat controlled by reducing the span lengths and the dead loads since the size of the steel section is greatly affected by these two parameters.

The sequence and timing of the slab pours during the construction of a continuous composite bridge, both prestressed and nonprestressed, must be considered during design. Some sections of the slab may be subjected to significant stresses due to creep and shrinkage as well as dead load stresses from portions of the slab poured at a later date.

Finally, as shown in Fig. 22, the stress range in the tension flange of the steel beam (Region 3) due to live load plus impact has been reduced to a level where fatigue of the flange is no longer a problem. This stress range is approximately 3 ksi for both types of prestressed continuous composite beams and is well within the allowable range of 13.0 ksi permitted by AASHTO for 2,000,000 cycles of load when shear connectors are welded to the tension flange.[7] Considering the less than favorable slab to steel section ratio of the beams in the design examples, it is obvious that for larger ratios fatigue of the top flange can easily be prevented by prestressing the slab in the negative moment region.

6. SUMMARY AND CONCLUSIONS

A pilot investigation was undertaken to determine the feasibility of prestressing the slab in the negative moment region of continuous composite bridge beams. Two simply supported composite steel-concrete beams with prestressed slabs were used to simulate the prestressed negative moment region of a two-span continuous composite beam. In one beam the prestress force was applied prior to making the shear connection while in the other the prestress force was applied after the shear connection was made. The beams were tested under static loading and the results of those tests are presented herein.

The analysis of these results shows that prestressing the slab in the negative moment region is a feasible technique that can be used to: (1) prevent slab cracking up to a level of live load plus impact; (2) eliminate consideration of the fatigue problem in the tension flange of the steel beam due to the presence of welded shear connectors; and (3) make the full transformed section in the negative moment region effective in resisting live loads as in the positive moment region.

Although further research is desirable, an analysis of the test results indicates that existing AASHTO specifications and design procedures can be used to design prestressed continuous composite bridges if some minor modifications are made. These modifications include the use of the transformed area method for computing section properties in the negative moment region when the slab is adequately prestressed and consider the resulting increased slab force in the design of the shear connection for both fatigue and ultimate strength requirements.

Design examples are presented for continuous composite bridge beams in three categories:
(1) nonprestressed, (2) prestressed before shear connection and (3) prestressed after shear connection. These examples illustrate the problems encountered in the design of continuous composite bridges and provide the basis for an economic comparison for the three categories of bridge beams.

A study of the test results, the design examples and the economic comparison leads to the following major conclusions:

(1) Considering the competitive construction cost, improved structural behavior and probable reduced maintenance and repair costs over the service life of the structure, continuous composite bridges with slab prestressing in the negative moment region are a desirable economic alternative to conventional nonprestressed continuous composite bridges.

(2) The application of the prestressing force after the shear connection has been made is more desirable than the application before shear connection. The cost of the required additional prestressing is low and substantial benefits can be obtained through stress reductions and lighter sections in all spans.

(3) The use of higher levels of prestress should be considered to maximize the benefits obtained through the use of prestressing. Not only is a factor of safety against slab cracking provided, but when the slab is prestressed after shear connection, substantial material and cost savings can result from the use of smaller beam sections in both the positive and negative moment regions.

(4) Finally, further research is required to develop more comprehensive design recommendations. This research should investigate the parameters governing the use of grouped shear connectors, the optimum level of prestress and the fatigue behavior of all components of prestressed continuous composite bridges.

APPENDICES

APPENDIX A

DESIGN OF NONPRESTRESSED CONTINUOUS COMPOSITE BRIDGE BEAM

Design Parameters

Interior Bridge Beam - effective width of slab = 60 in.

Loading: HS20-44 truck and lane loading considered

Dead Load - 863 plf

Fatigue - design for 2,000,000 cycles of load

Concrete: f'_c = 4000 psi

n = 8

Steel: Beams and Coverplates - F_y = 36 ksi

Reinforcing Steel - F_y = 40 ksi

Shear Connection: 4 in. high, 3/4 in. diameter stud shear connectors

Maximum Positive Moments:

Region 1: M_{DL} = 306 kip-ft M_{LL} = 565 kip-ft

Region 2: M_{DL} = 624 kip-ft M_{LL} = 638 kip-ft

Region 3: M_{DL} = 210 kip-ft M_{LL} = 400 kip-ft

Maximum Negative Moments:

Region 1: M_{DL} = 50 kip-ft M_{LL} = -270 kip-ft

Region 2: M_{DL} = 220 kip-ft M_{LL} = -160 kip-ft

Region 3: M_{DL} = -930 kip-ft M_{LL} = -651 kip-ft

Transformed width of slab = $\frac{60}{8}$ = 7.5 in.

Transformed area of slab = 7.5 x 7 = 52.5 in.^{2}

Region 1

Try W33 x 141: $A_s = 41.6 \text{ in.}^2$ $d_s = 33.31$ in.

$I_s = 7460 \text{ in.}^4$ $S_s = 448 \text{ in.}^3$

Properties of Composite Section
(Computed using transformed area method)

$y_b = 27.90$ in. $y_t = 12.41$ in.

$A_T = 94.1 \text{ in.}^2$ $I_T = 17{,}100 \text{ in.}^4$

Check Stresses

Maximum Positive Moment

$$\sigma_{tc} = \frac{565 \times 12 \times 12.41}{17{,}100 \times 8} = 615 \text{ psi(C)} < 1600 \text{ psi} \quad \underline{\text{OK}}$$

$$\sigma_{bc} = \frac{306 \times 12}{448} + \frac{565 \times 12 \times 27.9}{17{,}100}$$

$$\sigma_{bc} = 8.2 + 11.05 = 19.25 \text{ ksi(T)} < 20.0 \text{ ksi} \quad \underline{\text{OK}}$$

Maximum Negative Moment

$$\sigma_{tc} = \frac{270 \times 12 \times 12.41}{17{,}100 \times 8} = 294 \text{ psi(T)}$$

Note: Reference 6 requires that the allowable tensile stress for concrete is zero. Under the numerous L.L. configurations possible in continuous bridges, there always exists one for each cross section that will result in a L.L. negative moment at that section (see Fig. 18). Thus, there is a L.L. configuration for every cross section which will result in a tensile stress condition in the slab which should be considered. For the sections in Region 3 (negative moment region defined by dead load) this tensile stress will either be eliminated through the use of prestressing or the slab will not be considered for live load. In Regions 1 and 2 (positive moment regions) several alternatives exist. In most cases, the tensile stress is small and can be ignored. As illustrated below, the steel beam alone can carry both the D.L. and L.L. moment.

$$\sigma_{bs} = \frac{50 \times 12}{448} - \frac{270 \times 12}{448}$$

$$\sigma_{bs} = 1.34 - 7.23 = 5.89 \text{ ksi(C)} \qquad \underline{\text{OK}}$$

In addition, since the tensile stress of 4000 psi concrete lies between 300 psi and 400 psi, the slab in this example should not crack under the negative moment. If the tensile stress is too large to be ignored, the slab prestressing can be extended as required.

$$\sigma_{bc} = \frac{50 \times 12}{448} - \frac{270 \times 12 \times 27.9}{17{,}100}$$

$$\sigma_{bc} = 1.34 - 5.29 = 3.95 \text{ ksi(C)} \qquad \underline{\text{OK}}$$

Region 2

Try W33 x 200: $A_s = 58.9 \text{ in.}^2$ $d_s = 33.0 \text{ in.}$

$I_s = 11{,}100 \text{ in.}^4$ $S_s = 671 \text{ in.}^3$

Properties of Composite Section

$y_b = 25.9 \text{ in.}$ $y_t = 14.1 \text{ in.}$

$A_T = 111.4 \text{ in.}^2$ $I_T = 22{,}420 \text{ in.}^4$

Check Stresses

Maximum Positive Moment

$$\sigma_{tc} = \frac{638 \times 12 \times 14.1}{22{,}420 \times 8} = 600 \text{ psi(C)} < 1600 \text{ psi} \quad \underline{\text{OK}}$$

$$\sigma_{bc} = \frac{624 \times 12}{671} + \frac{638 \times 12 \times 25.9}{22{,}420}$$

$$\sigma_{bc} = 11.16 + 8.84 = 20.0 \text{ ksi(T)} = 20.0 \text{ ksi} \quad \underline{\text{OK}}$$

Maximum Negative Moment

$$\sigma_{tc} = \frac{160 \times 12 \times 14.1}{22{,}420 \times 8} = 157 \text{ psi(T)}$$

Note: The discussion of concrete tensile stress for Region 1 also applies to Region 2.

$$\sigma_{bc} = \frac{220 \times 12}{671} - \frac{160 \times 12 \times 25.9}{22{,}420}$$

$$\sigma_{bc} = 3.94 - 2.22 = 1.72 \text{ ksi(T)} \quad \underline{\text{OK}}$$

Region 3 - Negative Moment Region

Try W33 x 200 with ½ in. thick, 15 in. wide coverplate on top and 3/4 in. thick, 15 in. wide coverplate on bottom. Consider slab reinforcement to be effective in resisting live loads. Assume reinforcement equal to 1% of the cross-sectional area of the slab and acting at mid-depth.

Properties of Steel Beam with Coverplates

y_{bs} = 16.4 in. y_{ts} = 17.85 in.

A_{ss} = 77.65 in.^{2} I_{ss} = 16,355 in.^{4}

Properties of Composite Steel Section
(Steel Beam, Coverplates and Reinforcement)

y_b = 17.5 in. y_t = 23.75 in.

A_{sc} = 81.85 in.^{2} I_{sc} = 18,170 in.^{4}

Compute Allowable Compressive Stress

The allowable compressive stress for steel is governed by the following equation from Table 1.7.1 of the AASHTO Specifications (Ref. 6).

$$\sigma_{all} = 20{,}000 - 7.5\left(\frac{l}{b}\right)^2$$

where: l = unsupported length in inches

b = width of bottom flange and coverplates

By Note 1 to Table 1.7.1: σ_{all} may be increased by 20% but may not exceed 20 ksi.

By Note 2 to Table 1.7.1: l = length of D.L. Negative Moment Region (use 30' = 360" for this problem).

$$\sigma_{all} = 20{,}000 - 7.5\left(\frac{360}{15}\right)^2 \times 1.20$$

$$\sigma_{all} = (20{,}000 - 4320) \times 1.20$$

$$\sigma_{all} = 18.8 \text{ ksi}$$

Check Stresses

Maximum Positive Moment

$$\sigma_{ts} = \frac{210 \times 12 \times 17.85}{16{,}355} + \frac{400 \times 12 \times 16.75}{18{,}170}$$

$$\sigma_{ts} = 2.75 + 4.42 = 7.2 \text{ ksi(C)} < 18.8 \text{ ksi} \qquad \underline{\text{OK}}$$

Maximum Negative Moment

$$\sigma_{ts} = \frac{930 \times 12 \times 17.85}{16{,}355} + \frac{651 \times 12 \times 16.75}{18{,}170}$$

$$\sigma_{ts} = 12.2 + 7.2 = 19.4 \text{ ksi(T)} < 20.0 \text{ ksi} \qquad \underline{\text{OK}}$$

$$\sigma_{bs} = \frac{930 \times 12 \times 16.4}{16{,}355} + \frac{651 \times 12 \times 17.5}{18{,}170}$$

$$\sigma_{bs} = 11.2 + 7.5 = 18.7 \text{ ksi(C)} < 18.8 \text{ ksi} \qquad \underline{\text{OK}}$$

$$\sigma_{tr} = \frac{651 \times 12 \times 21.75}{18{,}170} = 9.35 \text{ ksi(T)} < 20.0 \text{ ksi} \qquad \underline{\text{OK}}$$

Reinforcement

Regions 1 and 2

Neither the transverse nor longitudinal reinforcement will be designed since they will be the same for all three designs and will add nothing to the comparison of the designs. The transverse or main reinforcement must be adequate to distribute the loads across the steel girders. The longitudinal reinforcement must be adequate to control temperature and shrinkage and to distribute the concentrated wheel loads along the length of the slab. In most designs, the longitudinal reinforcement will be designed as a percentage of the transverse reinforcement.

Region 3

As for regions 1 and 2, the transverse reinforcement will not be designed for region 3. The longitudinal reinforcement, however, is pertinent and will be discussed.

Reference 7 specifies the use of a minimum steel percentage equal to 1% of the cross-sectional area of the concrete slab in the negative moment region. In addition, it is required that two-thirds of this be placed in the top layer.

$p = 0.01 \times 60 \times 7 = 4.2 \text{ in.}^2$

Top layer = $2/3 \times 4.2 = 2.8 \text{ in.}^2$

Use 5 #7 bars 12' o.c. = 3.0 in.^2 OK

Bottom layer = $1/3 \times 4.2 = 1.4 \text{ in.}^2$

Use 5 #5 bars 12' o.c. = 1.55 in.^2 OK

Shear Connection

Allowable Stresses for Stud Connectors

Fatigue: $Z_r = \propto d^2$ (when $H/d \geq 4$)

$$H/d = \frac{4.0}{0.75} = 5.33$$

$$Z_r = 7850 \times (0.75)^2 = 4.42 \text{ kips}$$

Using two connectors per cross section:

$$\Sigma Z_r = 2 \times 4.42 = 8.84 \text{ kips}$$

Static: $E_c = 33\ w^{3/2} \sqrt{f'_c}$

$$E_c = 33 \times (150)^{3/2} \sqrt{4000}$$

$$E_c = 3{,}840{,}000 \text{ psi}$$

$$S_u = 0.0004\ d^2 \sqrt{f'_c E_c}$$

$$S_u = 0.0004 \times (0.75)^2 \sqrt{3{,}840{,}000 \times 4000}$$

$$S_u = 27.8 \text{ kips}$$

Due to symmetry, the shear connection will be designed for one-half of the structure and the results will be doubled. Since boundaries for the static criteria are governed by points of maximum moment and not dead load points of contraflexure, the shear connection cannot be designed according to regions 1, 2 and 3; however, the portions of these regions included between the points of maximum moment will be listed.

End Support to Maximum Moment in Region 1

(First 33 ft. of Region 1)

Fatigue: $V_r = 29.5$ kips $\quad I = 17{,}100 \text{ in.}^4$

$$Q = 468 \text{ in.}^3$$

$$S_r = \frac{V_r\ Q}{I} = \frac{29.5 \times 468}{17{,}100} = 0.8 \text{ k/in.}$$

$$s = \frac{8.84}{0.8} = 11.05 \text{ in.} \quad \text{use 11 in. o.c.}$$

$$N = \frac{33 \times 12 \times 2}{11} = 72 \text{ connectors}$$

Static: $P_1 = As\ \ Fy$

$$P_1 = 41.6 \times 36 = 1498 \text{ kips}$$

$$P_2 = 0.85\ f'_c\ bc$$

$$P_2 = 0.85 \times 4.0 \times 60 \times 7 = 1430 \text{ kips}$$

$$N_1 = \frac{P}{\phi\ S_u} = \frac{1430}{0.85 \times 27.8}$$

$$N_1 = 61 \text{ connectors}$$

Fatigue governs: use 72 stud connectors in pairs @ 11 in. o.c.

Maximum Moment in Region 1 to Interior Support

(Last 17 ft. of Region 1 and first 30 ft. of Region 3)

Fatigue: Last 17 ft. of Region 1

$$N = \frac{17 \times 12 \times 2}{11} = 37 \text{ connectors}$$

First 30 ft. of Region 3

$V_r = 36.0$ kips $I = 18{,}170$ in.^{4}

$Q = 85$ in.^{3}

$$S_r = \frac{36.0 \times 85}{18{,}170} = 0.17 \text{ k/in.}$$

$$s = \frac{8.84}{0.17} = 52 \text{ in.}$$

$$N = \frac{30 \times 12 \times 2}{52} = 14 \text{ connectors}$$

Total N = 37 + 14 = 51 connectors

Static: $P_1 = 41.6 \times 36 = 1498$ kips

$P_2 = 1430$ kips

$P_3 = A_s^r \; F_y^r$

$P_3 = 4.55 \times 40 = 182$ kips

$$N_2 = \frac{P + P_3}{\phi S_u}$$

$$N_2 = \frac{1430 + 182}{0.85 \times 27.8} = 68 \text{ connectors}$$

$$s = \frac{47 \times 12 \times 2}{68} = 16.6 \text{ in. use 16 in. o.c.}$$

$$N = \frac{47 \times 12 \times 2}{16} = 70 \text{ connectors}$$

Static governs: use 70 stud connectors in pairs @ 16 in. o.c.

Note: Reference 7 permits stud connectors to be welded to the top flange throughout the entire negative moment region provided the stress range in the top flange due to live load plus impact is less than or equal to 13 ksi for 2,000,000 cycles of load. As shown below the stress range in the top flange in this design is 8.01 ksi which is less than 13 ksi; therefore, stud connectors can be placed continuously throughout the negative moment region to provide improved structural behavior of the composite section in this region.

$$f_r = \frac{(651 + 73) \times 12 \times 16.75}{18{,}170} = 8.01 \text{ ksi} \qquad \underline{\text{OK}}$$

Interior Support to Maximum Moment in Region 2

(Last 30 ft. of Region 3 and first 30 ft. of Region 2)

Fatigue: Last 30 ft. of Region 3

$V_r = 37.3$ kips $\quad I = 18{,}170$ in.^{4}

$Q = 85$ in.^{3}

$$S_r = \frac{37.3 \times 85}{18{,}170} = 0.174 \text{ k/in.}$$

$$s = \frac{8.84}{0.174} = 50.7 \text{ in. use } 50 \text{ in.}$$

$$N = \frac{30 \times 12 \times 2}{50} = 15 \text{ connectors}$$

<u>First 30 ft. of Region 2</u>

$V_r = 27.6$ kips $I = 22{,}420$ in.^{4}

$Q = 557$ in.^{3}

$$S_r = \frac{27.6 \times 557}{22{,}420} = 0.69 \text{ k/in.}$$

$$s = \frac{8.84}{0.69} = 12.8 \text{ in.} \quad \text{use } 12\ 3/4 \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{12.75} = 58 \text{ connectors}$$

Total $N = 58 + 15 = 73$ connectors

<u>Static</u>: $P_1 = 58.9 \times 36 = 2120$ kips

$P_2 = 1430$ kips

$P_3 = 182$ kips

$N_2 = 68$ connectors

<u>Fatigue governs</u>:

<u>Last 30 ft. of Region 3</u>

Maximum connector spacing permitted by Ref. 6 is 24 in.

Try staggered singles @ 24 in. o.c.

$$N = \frac{30 \times 12}{24} = 15 \text{ connectors}$$ <u>OK</u>

Use 15 staggered single stud connectors @ 24 in. o.c.

First 30 ft. of Region 2

$$s = \frac{30 \times 12 \times 2}{58} = 12.42 \text{ in.} \quad \text{use } 12\tfrac{1}{2} \text{ in. o.c.}$$

Use 58 stud connectors in pairs @ $12\frac{1}{2}$ in. o.c.

Total Stud Shear Connectors for Beam

First 33 ft. of Region 1	72
Last 17 ft. of Region 1 and First 30 ft. of Region 3	70
Last 30 ft. of Region 3	15
First 30 ft. of Region 2	58
Subtotal	215
	x 2
Total	430

APPENDIX B

DESIGN OF PRESTRESSED CONTINUOUS COMPOSITE BRIDGE BEAM (SLAB PRESTRESSED BEFORE SHEAR CONNECTION)

Design Parameters

Identical to those in Appendix A with the following additions:

Prestressing Steel: Regular Grade - $F_y = 130$ ksi

$F_u = 145$ ksi

Special Grade - $F_y = 140$ ksi

$F_u = 160$ ksi

Prestressing Loss: Total due to all causes = 15%

Region 1

Identical to Appendix A.

Region 2

Identical to Appendix A.

Region 3

Steel beam carries D.L.

Full Transformed Composite Section carries LL + I.

Try W33 x 200 with 15 in. wide, 3/4 in. thick coverplate on the bottom.

Properties of Steel Beam with Coverplate

y_{bs} = 14.54 in. y_{ts} = 19.21 in.

A_{ss} = 70.15 in.^{2} I_{ss} = 13,790 in.^{4}

Properties of Composite Section

y_b = 24.26 in. y_t = 16.49 in.

A_T 122.65 in.^{2} I_T = 29,490 in.^{4}

Allowable Compressive Stress

Identical to Appendix A (σ_{all} = 18.8 ksi)

Check Stresses

Maximum Positive Moment

$$\sigma_{tc} = \frac{400 \times 12 \times 16.49}{29{,}490 \times 8} = 336 \text{ psi(C)} < 1600 \text{ psi} \quad \underline{\text{OK}}$$

$$\sigma_{bc} = \frac{210 \times 12 \times 14.54}{13{,}790} + \frac{400 \times 12 \times 24.26}{29{,}490}$$

$$\sigma_{bc} = 2.66 + 3.95 = 6.61 \text{ ksi(T)} < 20.0 \text{ ksi} \quad \underline{\text{OK}}$$

Maximum Negative Moment

$$\sigma_{tc} = \frac{651 \times 12 \times 16.49}{29{,}490 \times 8} = 546 \text{ psi(T)}$$

$$\sigma_{bc} = \frac{930 \times 12 \times 14.54}{13{,}790} + \frac{651 \times 12 \times 24.26}{29.490}$$

$$\sigma_{bc} = 11.77 + 6.43 = 18.2 \text{ ksi(C)} < 18.8 \text{ ksi} \quad \underline{\text{OK}}$$

Note: The actual stress in the top of the slab will be controlled through the application of a prestressing force.

This will be covered in the design of the Prestressing Steel.

Reinforcement

Regions 1 and 2

Identical to Appendix A.

Region 3

Since this region will be prestressed, there should be no need to use any longitudinal reinforcement other than that required to apply the prestress force (designed in the next section) and any required to tie the transverse reinforcement. However, should a long delay be required between the casting of the slab and the post-tensioning operation, additional reinforcement should be provided to control the effects of temperature and shrinkage.

Prestressing Steel

The amount of prestressing steel required depends upon the load level up to which cracking is to be prevented. In this design and that contained in Appendix C, two levels of prestress will be considered. The first will prevent cracking up to working load and the second will provide a factor of safety of 1.5 against cracking under working load.

For both cases, the limiting tensile stress will be zero. Prestress loss will be taken as 15% as recommended in Ref. 17. The high strength steel bars used for the post-tensioning will be designed based on design values contained in Ref. 18.

<u>No Slab Cracking at Working Load</u>

$$F_f = \sigma_{tc} \quad A_c$$

$$F_f = 546 \times 7 \times 60$$

$$F_f = 229 \text{ kips}$$

$$F_i = \frac{229}{0.85} = 270 \text{ kips}$$

Use four 1 in. diameter Regular grade prestressing bars @ 15 in. o.c.

Maximum Recommended F_f = 272 kips <u>OK</u>

Maximum Recommended F_i = 320 kips <u>OK</u>

<u>No Slab Cracking at 1.5 x Working Load</u>

$$F_f = 1.5 \quad \sigma_{tc} \quad A_c$$

$$F_f = 1.5 \times 229 = 344 \text{ kips}$$

$$F_i = \frac{344}{0.85} = 405 \text{ kips}$$

Use four 1 1/8 in. diameter Regular grade prestressing bars @ 15 in. o.c.

Maximum Recommended F_f = 348 kips <u>OK</u>

Maximum Recommended F_i = 405 kips OK

Shear Connection

Z_r, $\sum Z_r$ and S_u are identical to those used in Appendix A.

End Support to Maximum Moment in Region 1

(First 33 ft. of Region 1)

Identical to Appendix A.

Fatigue governs: use 72 stud connectors in pairs @ 11 in. o.c.

Maximum Moment in Region 1 to Interior Support

(Last 17 ft. of Region 1 and first 30 ft. of Region 3)

Fatigue: Last 17 ft. of Region 1

Identical to Appendix A.

$N = 37$ connectors

First 30 ft. of Region 3

$V_r = 36.0$ kips $\quad I = 29,490 \text{ in.}^4$

$Q = 52.5 \times 12.99 = 682 \text{ in.}^3$

$S_r = \dfrac{36.0 \times 682}{29,490} = 0.84$ k/in.

$s = \dfrac{8.84}{0.84} = 10.52$ in. use $10\frac{1}{2}$ in. o.c.

$N = \dfrac{30 \times 12 \times 2}{10.5} = 69$ connectors

Total $N = 37 + 69 = 106$ connectors

Static: $P_1 = 41.6 \times 36 = 1498$ kips

$P_2 = 1430$ kips

$P_3 = 0$

$P_4 = A_s^p \quad F_y^p$

No Cracking at Working Load

$P_4 = 4(0.785)\ 130 = 408$ kips

$$N_2 = \frac{P + P_3 + P_4}{\phi\ S_u}$$

$$N_2 = \frac{1430 + 0 + 408}{0.85 \times 27.8} = 78 \text{ connectors}$$

No Cracking at 1.5 x Working Load

$P_4 = 4(0.994)\ 130 = 517$ kips

$$N_2 = \frac{1430 + 0 + 517}{0.85 \times 27.8} = 82 \text{ connectors}$$

Fatigue governs for both prestress levels:

Last 17 ft. of Region 1

$$s = \frac{17 \times 12 \times 2}{38} = 10.72 \text{ in.} \quad \text{use } 10\ 3/4 \text{ in. o.c.}$$

Use 38 stud connectors in pairs @ 10 3/4 in. o.c.

First 30 ft. of Region 3

$$s = \frac{30 \times 12 \times 2}{70} = 10.29 \text{ in.} \quad \text{use } 10\tfrac{1}{4} \text{ in. o.c.}$$

Use 70 stud connectors in pairs @ $10\frac{1}{4}$ in. o.c.

Interior Support to Maximum Moment in Region 2

(Last 30 ft. of Region 3 and first 30 ft. of Region 2)

Fatigue: Last 30 ft. of Region 3

$$V_r = 37.3 \text{ kips} \qquad I = 29{,}490 \text{ in.}^4$$

$$Q = 682 \text{ in.}^3$$

$$S_r = \frac{37.3 \times 682}{29{,}490} = 0.87 \text{ k/in.}$$

$$s = \frac{8.84}{0.87} = 10.15 \text{ in.} \quad \text{use } 10\ 1/8 \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{10.125} = 71 \text{ connectors}$$

First 30 ft. of Region 2

Identical to Appendix A.

$$N = 58 \text{ connectors}$$

Total $N = 71 + 58 = 129$ connectors

Static: $P_1 = 58.9 \times 36 = 2120$ kips

$$P_2 = 1430 \text{ kips}$$

$$P_3 = 0$$

No Cracking at Working Load

P_4 = 408 kips

N_2 = 78 connectors

No Cracking at 1.5 x Working Load

P_4 = 517 kips

N_2 = 82 connectors

Fatigue governs for both prestress levels:

Last 30 ft. of Region 3

$$s = \frac{30 \times 12 \times 2}{72} = 10 \text{ in. o.c.}$$

Use 72 stud connectors in pairs @ 10 in. o.c.

First 30 ft. of Region 2

Use 58 stud connectors in pairs @ $12\frac{1}{2}$ in. o.c.

Total Stud Shear Connectors for Beam

First 33 ft. of Region 1	72
Last 17 ft. of Region 1	38
First 30 ft. of Region 3	70
Last 30 ft. of Region 3	72
First 30 ft. of Region 2	58
Subtotal	310
	x 2
Total	620

APPENDIX C

DESIGN OF PRESTRESSED CONTINUOUS COMPOSITE BRIDGE BEAM (SLAB PRESTRESSED AFTER SHEAR CONNECTION)

Design Parameters

Identical to Appendix B.

Regions 1 and 2

Since the prestress force affects the stresses in these regions, Region 3 will be designed first and the required prestress force will be determined. Regions 1 and 2 can then be designed taking into account the effect of this prestress force.

Region 3

Steel beam carries D.L.

Full Transformed Composite Section carries LL + I.

Try W33 x 200 with 15 in. wide, $\frac{1}{4}$ in. thick coverplate on the bottom.

Properties of Steel Beam with Coverplate

y_{bs} = 15.75 in. $\qquad$ y_{ts} = 17.5 in.

A_{ss} = 62.65 in.^{2} $\qquad$ I_{ss} = 12,074 in.^{4}

Properties of Composite Section

$y_b = 25.32$ in. $\quad y_t = 14.93$ in.

$A_T = 115.15$ in.^{2} $\quad I_T = 24{,}885$ in.^{4}

Allowable Compressive Stress

Identical to Appendix A ($\sigma_{all} = 18.8$ ksi)

Check Stresses

Maximum Positive Moment

$$\sigma_{tc} = \frac{400 \times 12 \times 14.93}{24{,}885 \times 8} = 360 \text{ psi(C)} < 1600 \text{ psi } \underline{\text{OK}}$$

$$\sigma_{bc} = \frac{210 \times 12 \times 15.75}{12{,}074} + \frac{400 \times 12 \times 25.32}{24{,}885}$$

$$\sigma_{bc} = 3.29 + 4.88 = 8.17 \text{ ksi(T)} < 20.0 \text{ ksi} \quad \underline{\text{OK}}$$

Maximum Negative Moment

$$\sigma_{tc} = \frac{651 \times 12 \times 14.93}{24{,}885 \times 8} = 586 \text{ psi(T)} > 0 \text{ ksi}$$

$$\sigma_{bc} = \frac{930 \times 12 \times 15.75}{12{,}074} + \frac{651 \times 12 \times 25.32}{24{,}885}$$

$$\sigma_{bc} = 14.56 + 7.95 = 22.51 \text{ ksi(C)} > 18.8 \text{ ksi}$$

Note: The actual stress in the top of the slab and at the bottom of the steel section will be controlled through the application of a prestressing force. This will be covered in the design of the Prestressing Steel.

Reinforcement

Regions 1 and 2

Identical to Appendix A.

Region 3

Identical to Appendix B.

Prestressing Steel

As illustrated in Appendix D and Fig. 24, the stress above the neutral axis at a cross section in Region 3 due to an applied prestress force (F) is determined by the following equation:

$$\sigma_{ps} = \frac{-F}{A_T} - \frac{(Fe)c}{I_T} + \frac{Mc}{I_T} + \frac{Vhc}{I_T}$$

By placing the prestressing bars at the mid-depth of the slab: e = 15.7 - 3.5 = 12.2 in. = 1.02 ft. Values for M and V in terms of F and e were computed in Appendix D and are as follows:

$M = 0.392Fe \qquad V = 0.00784Fe$

where: F is in kips

e is in feet

Using the moment diagram shown in Fig. 24c, the last three terms in the above equation can be combined resulting in the following equation for the stress over the interior support due to the applied prestress force.

$$\sigma_{ps} = \frac{-F}{A_T} - \frac{(0.373Fe)\ c}{I_T}$$

No Slab Cracking at Working Load

$$\sigma_{tc} + \sigma_{ps} = 0$$

$$0 = \sigma_{tc} + \left(\frac{-F}{115.15} - \frac{0.373F \times 1.02 \times 12 \times 14.93}{24,885}\right)$$

$$\sigma_{tc} = 0.0114F$$

$$F_f = \frac{\sigma_{tc}}{0.0114}$$

$$F_f = \frac{586 \times 8}{0.0114} = 410 \text{ kips}$$

$$F_i = \frac{410}{0.85} = 483 \text{ kips}$$

Use four $1\frac{1}{4}$ in. diameter Regular grade prestressing bars @ 15 in. o.c.

Maximum Recommended F_f = 428 kips <u>OK</u>

Maximum Recommended F_i = 500 kips <u>OK</u>

<u>Check σ_{bc}</u>

$$\sigma_{bc} = -22.51 + \frac{410}{115.15} + \frac{0.373 \times 410 \times 12.2 \times 25.32}{24,885}$$

$$\sigma_{bc} = -22.51 + 3.56 + 1.90 = 17.05 \text{ ksi(C)} < 18.8 \text{ ksi}$$

<u>OK</u>

<u>No Slab Cracking at 1.5 x Working Load</u>

Try a W33 x 200:

$$\sigma_{tc} = \frac{651 \times 12 \times 14.1}{22,420 \times 8} = 614 \text{ psi}$$

$$1.5\ \sigma_{tc} + \sigma_{ps} = 0$$

$$0 = 1.5\ \sigma_{tc} + \left(\frac{-F}{111.4} - \frac{0.373F \times 1.02 \times 12 \times 14.1}{22,420}\right)$$

$$1.5\ \sigma_{tc} = 0.01185F$$

$$F_f = \frac{1.5 \times 614 \times 8}{0.1185} = 622 \text{ kips}$$

$$F_i = \frac{622}{0.85} = 732 \text{ kips}$$

Use five 1 3/8 in. diameter Regular grade prestressing bars @ 12 in. o.c.

Maximum Recommended F_f = 645 kips

Maximum Recommended F_i = 755 kips

<u>Check σ_{bc}</u>

$$\sigma_{bc} = \frac{-930 \times 12}{671} - \frac{651 \times 12 \times 25.9}{22{,}420} + \frac{622}{111.4} + \frac{0.373 \times 622 \times 12.2 \times 25.9}{22{,}420}$$

$$\sigma_{bc} = -16.6 - 9.0 + 5.6 + 3.3 = 16.7 \text{ ksi(C)} < 18.8 \text{ ksi}$$

<u>OK</u>

The composite beam sections for regions 1 and 2 can now be designed using the applied prestress force.

<u>Region 1</u>

From Appendix D and Fig. 24, the stress below the neutral axis at a cross section in Region 1 due to the prestressing is:

$$\sigma_{ps} = \frac{-0.00784 Fe \times L \times c}{I_T}$$

Try next lightest W33 section; that is, a W33 x 130.

<u>Check Stress in Bottom Flange</u>

<u>For F = 410 kips</u>

$$\sigma_{bc} = \frac{306 \times 12}{406} + \frac{565 \times 12 \times 28.14}{15,826} - \frac{0.00784(410)(1.02)(33\times12)(28.14)}{15,826}$$

$$\sigma_{bc} = 9.04 + 12.05 - 2.31 = 18.78 \text{ ksi(T)}$$
$$< 20.0 \text{ ksi} \qquad \underline{\text{OK}}$$

<u>Use W33 x 130</u>

<u>For F = 622 kips</u>

Try lightest W33 section; that is, a W33 x 118.

$$\sigma_{bc} = \frac{306 \times 12}{359} + \frac{565 \times 12 \times 28.42}{14,427} - \frac{3.17(622)28.42}{14,427}$$

$$\sigma_{bc} = 10.23 + 13.36 - 3.88 = 19.7 \text{ ksi(T)}$$
$$< 20.0 \text{ ksi} \qquad \underline{\text{OK}}$$

<u>Use W33 x 118</u>

<u>Region 2</u>

From Appendix D and Fig. 24, the stress below the neutral axis at a cross section in Region 2 due to the prestressing is:

$$\sigma_{ps} = \frac{0.627Fe \times c}{I_T}$$

Try the next lightest W33 section; that is, a W33 x 152.

<u>Check Stress in Bottom Flange</u>

<u>For F = 410 kips</u>

$$\sigma_{bc} = \frac{624 \times 12}{487} + \frac{638 \times 12 \times 23.9}{19,674} - \frac{0.627(410)(12.2)(23.9)}{19,674}$$

$$\sigma_{bc} = 15.38 + 9.3 - 3.8 = 20.88 \text{ ksi} > 20.0 \text{ ksi}$$

<u>NG</u>

<u>Use W33 x 200</u>

<u>For F = 622 kips</u>

$$\sigma_{bc} = 24.68 - \frac{0.627(622)(12.2)(23.9)}{19,674}$$

$$\sigma_{bc} = 24.67 - 5.77 = 18.90 \text{ ksi(T)} < 20.0 \text{ ksi} \quad \underline{\text{OK}}$$

<u>Use W33 x 152</u>

<u>Shear Connection</u>

Z_r, ΣZ_r and S_u are identical to those found in Appendix A.

<u>End Support to Maximum Moment in Region 1</u>

(First 33 ft. of Region 1)

<u>No Cracking at Working Load</u>

<u>Fatigue</u>: $V_r = 29.5$ kips $\quad I = 15,826 \text{ in.}^4$

$$Q = 444 \text{ in.}^3$$

$$S_r = \frac{V_r Q}{I} = \frac{29.5 \times 444}{15,826} = 0.83 \text{ k/in.}$$

$$s = \frac{8.84}{0.83} = 10.65 \text{ in.} \quad \text{use } 10\tfrac{1}{2} \text{ in. o.c.}$$

$$N = \frac{33 \times 12 \times 2}{10.5} = 76 \text{ connectors}$$

<u>Static</u>: $P_1 = 38.3 \times 36 = 1379$ kips

$$P_2 = 1430 \text{ kips}$$

$$N_1 = \frac{1379}{0.85 \times 27.8} = 59 \text{ connectors}$$

Fatigue governs: use 76 stud connectors in pairs @ $10\frac{1}{2}$ in. o.c.

No Cracking at 1.5 x Working Load

Fatigue: $V_r = 29.5$ kips $\qquad I = 14{,}427$ in.^{4}

$Q = 417$ in.^{3}

$$S_r = \frac{29.5 \times 417}{14{,}427} = 0.85$$

$$s = \frac{8.84}{0.85} = 10.36 \text{ in.} \qquad \text{use } 10\tfrac{1}{4} \text{ in. o.c.}$$

$$N = \frac{33 \times 12 \times 2}{10.25} = 78 \text{ connectors}$$

Static: $P_1 = 34.8 \times 36 = 1253$ kips

$P_2 = 1430$ kips

$$N_1 = \frac{1253}{0.85 \times 27.8} = 53 \text{ connectors}$$

Fatigue governs: use 78 stud connectors in pairs @ $10\frac{1}{4}$ in. o.c.

Maximum Moment in Region 1 to Interior Support

(Last 17 ft. of Region 1 and first 30 ft. of Region 3)

No Cracking at Working Load

Fatigue: Last 17 ft. of Region 1

$$N = \frac{17 \times 12 \times 2}{10.5} = 39 \text{ connectors}$$

First 30 ft. of Region 3

$V_r = 36.0$ kips $\qquad I = 24{,}885$ in.^{4}

$$Q = 600 \text{ in.}^3$$

$$S_r = \frac{36.0 \times 600}{24,885} = 0.87 \text{ k/in.}$$

$$s = \frac{8.84}{0.87} = 10.18 \text{ in.} \quad \text{use } 10\ 1/8 \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{10.125} = 72 \text{ connectors}$$

Total $N = 39 + 72 = 111$ connectors

<u>Static</u>: $P_1 = 1379$ kips

$P_2 = 1430$ kips

$P_3 = 0$

$P_4 = 4(1.227)(130) = 638$ kips

$$N_2 = \frac{1379 + 0 + 638}{0.85 \times 27.8} = 78 \text{ connectors}$$

<u>Fatigue governs</u>: <u>Last 17 ft. of Region 1 and First 30 ft. of Region 3</u>

Use same spacing for both regions

$$s = \frac{47 \times 12 \times 2}{112} = 10.07 \text{ in.} \quad \text{use 10 in. o.c.}$$

Use 112 stud connectors in pairs @ 10 in. o.c.

<u>No Cracking at 1.5 x Working Load</u>

<u>Fatigue</u>: <u>Last 17 ft. of Region 1</u>

$$N = \frac{17 \times 12 \times 2}{10.25} = 40 \text{ connectors}$$

<u>First 30 ft. of Region 3</u>

$V_r = 36.0$ kips $\qquad I = 22,420 \text{ in.}^4$

$$Q = 557 \text{ in.}^3$$

$$S_r = \frac{36.0 \times 557}{22,420} = 0.89 \text{ k/in.}$$

$$s = \frac{8.84}{0.89} = 9.88 \text{ in.} \quad \text{use } 9\ 7/8 \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{9.875} = 73 \text{ connectors}$$

Total $N = 40 + 73 = 113$ connectors

<u>Static</u>: $P_1 = 1253$ kips

$P_2 = 1430$ kips

$P_3 = 0$

$P_4 = 5(1.485)(130) = 965$ kips

$$N_2 = \frac{1253 + 0 + 965}{0.85 \times 27.8} = 94 \text{ connectors}$$

<u>Fatigue governs</u>: <u>Last 17 ft. of Region 1</u>

Use 40 stud connectors in pairs @ 10 1/8 in. o.c.

<u>First 30 ft. of Region 3</u>

$$s = \frac{30 \times 12 \times 2}{74} = 9.73 \text{ in.} \quad \text{use } 9\ 3/4 \text{ in. o.c.}$$

Use 74 stud connectors in pairs @ 9 3/4 in. o.c.

<u>Interior Support to Maximum Moment in Region 2</u>

(Last 30 ft. of Region 3 and first 30 ft. of Region 2)

<u>No Cracking at Working Load</u>

<u>Fatigue</u>: <u>Last 30 ft. of Region 3</u>

$V_r = 37.3$ kips $\qquad I = 24{,}885$ in.^{4}

$Q = 600$ in.^{3}

$$S_r = \frac{37.3 \times 600}{24{,}885} = 0.90 \text{ k/in.}$$

$$s = \frac{8.84}{0.90} = 9.82 \text{ in.} \quad \text{use } 9\ 3/4 \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{9.75} = 74 \text{ connectors}$$

<u>First 30 ft. of Region 2</u>

$V_r = 37.3$ kips $\qquad I = 22{,}420$ in.^{4}

$Q = 557$ in.^{3}

$$S_r = \frac{37.3 \times 557}{22{,}420} = 0.93 \text{ k/in.}$$

$$s = \frac{8.84}{0.93} = 9.54 \text{ in.} \quad \text{use } 9\tfrac{1}{2} \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{9.5} = 76 \text{ connectors}$$

Total $\quad N = 74 + 76 = 150$ connectors

<u>Static</u>: $\quad P_1 = 58.9 \times 36 = 2120$ kips

$P_2 = 1430$ kips

$P_3 = 0$

$P_4 = 638$ kips

$$N_2 = \frac{1430 + 0 + 638}{0.85 \times 27.8} = 88 \text{ connectors}$$

<u>Fatigue governs</u>: <u>Last 30 ft. of Region 3 and First 30 ft. of Region 2</u>

Use same spacing for both regions

$$s = \frac{60 \times 12 \times 2}{150} = 9.6 \quad \text{use } 9\ 5/8 \text{ in. o.c.}$$

Use 150 stud connectors in pairs @ 9 5/8 in. o.c.

No Cracking at 1.5 x Working Load

Fatigue: Last 30 ft. of Region 3

Same as Region 2 above.

$N = 76$ connectors

First 30 ft. of Region 2

$V_r = 37.3$ kips $\qquad I = 19{,}674 \text{ in.}^4$

$Q = 688 \text{ in.}^3$

$$S_r = \frac{37.3 \times 688}{19{,}674} = 1.30 \text{ k/in.}$$

$$s = \frac{8.84}{1.30} = 6.77 \text{ in.} \quad \text{use } 6\ 3/4 \text{ in. o.c.}$$

$$N = \frac{30 \times 12 \times 2}{6.75} = 107 \text{ connectors}$$

Total $N = 76 + 107 = 183$ connectors

Static: $P_1 = 44.8 \times 36 = 1613$ kips

$P_2 = 1430$ kips

$P_3 = 0$

$P_4 = 965$ kips

$$N_2 = \frac{1430 + 0 + 965}{0.85 \times 27.8} = 102 \text{ connectors}$$

Fatigue governs: Last 30 ft. of Region 3

Use 76 stud connectors in pairs @ $9\frac{1}{2}$ in. o.c.

First 30 ft. of Region 2

$$s = \frac{30 \times 12 \times 2}{10.8} = 6.67 \text{ in.} \quad \text{use } 6\ 5/8 \text{ in. o.c.}$$

Use 108 stud connectors in pairs @ 6 5/8 in. o.c.

Total Stud Shear Connectors for Beam

	Working Load	
	x 1.0	x 1.5
First 33 ft. of Region 1	76	78
Last 17 ft. of Region 1	40	40
First 30 ft. of Region 3	72	74
Last 30 ft. of Region 3	74	76
First 30 ft. of Region 2	76	108
Subtotal	338	376
	x 2	x 2
Total	676	752

APPENDIX D

VIRTUAL WORK ANALYSIS

In order to determine the effect of a prestressing force applied to the example bridge after the shear connection was made, the following analysis was required. The structure to be analyzed and the various loads on the primary structure are shown in Fig. 23. This figure is divided into the following five parts:

a. Structure to be analyzed and coordinate system used

b. Primary structure and moment due to applied prestressing force (F) at eccentricity (e)

c. Primary structure with unit load at point C and moment due to this loading

d. Primary structure with reaction at point C (R_c) applied as a load and the moment due to this loading

e. Primary structure with reaction at point F (R_F) applied as a load and the moment due to this loading.

For simplicity EI is assumed constant.

Using virtual work: $\Delta = \int_{l_1}^{l_2} \frac{m\ M\ dx}{EI}$

Compute Δ_{CF} (Deflection at point C due to prestress force (F) applied to primary structure)

$$\Delta_{CF} = \frac{Fe}{EI} \int_{50}^{80} \frac{5}{7} x \, dx + \frac{Fe}{EI} \int_{80}^{110} (-\frac{2}{7}x + 80)dx$$

$$+ \frac{Fe}{EI} \int_{170}^{230} (-\frac{2}{7}x + 80)dx$$

$$\Delta_{CF} = \frac{Fe}{EI} (1393 + 1586 + 1371)$$

$$\Delta_{CF} = 4350 \frac{Fe}{EI}$$

Compute Δ_{CR_c} (Deflection at point C due to reaction at C (R_c) applied as a load)

$$\Delta_{CR_c} = \frac{R_c}{EI} \int_{0}^{80} (\frac{5}{7}x) (\frac{5}{7}x)dx$$

$$+ \frac{R_c}{EI} \int_{80}^{280} (-\frac{2}{7}x + 80) \quad (-\frac{2}{7}x + 80)dx$$

$$\Delta_{CR_c} = \frac{R_c \times 10^4}{EI} (8.71 + 58.34 - 164.57 + 128.0)$$

$$\Delta_{CR_c} = 30.48 \times 10^4 \frac{R_c}{EI}$$

Compute Δ_{CR_F} (Deflection at point C due to reaction at F (R_F) applied as a load)

$$\Delta_{CR_F} = \frac{R_F}{EI} \int_{0}^{80} (\frac{5}{7}x)(\frac{2}{7}x)dx$$

$$+ \frac{R_F}{EI} \int_{80}^{200} (-\tfrac{2}{7}x + 80)(\tfrac{2}{7}x)dx$$

$$+ \frac{R_F}{EI} \int_{200}^{280} (-\tfrac{2}{7}x + 80)(-\tfrac{5}{7}x + 200)dx$$

$$\Delta_{CR_F} = \frac{R_F \times 10^4}{EI} (3.48-20.37+38.4+94.9-219.4+128)$$

$$\Delta_{CR_F} = 25.01 \times 10^4 \frac{R_F}{EI}$$

By superposition: $\Delta_C = \Delta_{CF} + \Delta_{CR_c} + \Delta_{CR_F}$

$$\Delta_C = 4350\frac{Fe}{EI} + 30.48 \times 10^4 \frac{R_c}{EI} + 25.01 \times 10^4 \frac{R_F}{EI}$$

Since compatibility requires that $\Delta_C = 0$ and symmetry requires that $R_C = R_F$, then:

$$0 = 4350\frac{Fe}{EI} + 55.49 \times 10^4 \frac{R_C}{EI}$$

$$R_C = \frac{4350Fe}{554,900}$$

$$R_C = 0.00784 \text{ Fe}$$

Note: Since all calculations were made using feet, e must also be in feet.

The reactions, shears and bending moments in the structure due to the prestressing force (F) are shown in

Figs. 24a, 24b, and 24c, respectively. A free body diagram of the negative moment region (Region 3) over the interior support at point C is shown in Fig. 24d. All stress resultants acting on the free body due to the prestressing force are shown in this figure and are used in the computation of the prestressing requirements for the design example in Appendix C.

APPENDIX E

COST COMPARISON

The construction costs of the three continuous composite bridge beams designed in Appendices A, B and C are compared below. For ease of comparison, the cost of one-half of the nonprestressed beam is considered to be the basic cost and is not computed. The construction costs of the two prestressed beams relative to the nonprestressed beam can then be illustrated through the use of additions to and deductions from the basic cost. All item costs are final in-place costs based on data listed in Ref. 15.

Design B - Continuous Composite Bridge Prestressed Before Shear Connection

Case 1 - Cracking Prevented to Working Load Level

Additions

190 stud connectors @ $0.80 EA = $152

Prestressing - 1 in. diameter Regular grade prestressing bars - 640 lb. @ $0.25 per lb. = $160

Total Additional Cost = $312

Deductions

No. 7 rebar - 300 LF = 613 lb.

No. 5 rebar - 300 LF = 314 lb.

Total rebar saved 927 lb. @ $0.39 per lb. = $362

Coverplates - 15 LF x 25.5 plf = 383 lb.

@ $0.35 per lb. = $134

Total Savings = $496

Net Cost Savings = $184

Case 2 - Cracking Prevented to 1.5 x Working Load Level

Additions

190 stud connectors @ $0.80 EA = $152

Prestressing - 1 1/8 in. diameter Regular grade prestressing bars - 810 lb. @ $0.23 per lb. = $186

Total Additional Cost = $338

Deductions

Same as Case 1. Total Savings = $529

Net Cost Savings = $191

Design C - Continuous Composite Bridge Prestressed After Shear Connection

Case 1 - Cracking Prevented to Working Load Level

Additions

246 stud connectors @ $0.80 EA = $197

Prestressing - $1\frac{1}{4}$ in. diameter Regular grade prestressing bars - 1000 lb. @ $0.23 per lb. = $230

Total Additional Cost = $427

Deductions

Rebar - same as Case 1 for Design B - $362

Reduced Beam Weight - Region 1 - 11 plf x 50 LF = 550 lb.

@ $0.35 per lb. = $196

Coverplates - 15 LF x 51 plf = 765 lb.@$0.35 per lb. = $268

Total Savings = $823

Net Cost Savings = $396

Case 2 - Cracking Prevented to 1.5 x Working Load Level

Additions

322 stud connectors @ $0.80 EA = $258

Prestressing - 1 3/8 in. diameter Regular grade prestressing bars - 1515 lb. @ $0.23 = $349

Total Additional Cost = $607

Deductions

Rebar - same as Case 1 for Design B = $362

Reduced Beam Weight - Region 1 - 23 plf x 50 LF = 1150 lb.

Region 2 - 48 plf x 30 LF = 1440 lb.

Total 2590 lb.

@ $0.35 per lb. = $907

Coverplates - 15 LF x 63.8 plf = $957

Total Savings = $2226

Net Cost Savings = $1619

APPENDIX F

NOTATION

A_c - cross-sectional area of concrete slab

A_s - cross-sectional area of steel beam

A_s^p - cross-sectional area of prestressing steel

A_s^r - cross-sectional area of longitudinal reinforcing steel

A_{sc} - cross-sectional area of composite steel section

A_{ss} - cross-sectional area of steel beam with coverplates

A_T - cross-sectional area of transformed composite beam section

C - compressive stress

E - modulus of elasticity

E_c - modulus of elasticity for concrete

F - unknown prestressing force

F_f - final prestressing force after losses

F_i - initial prestressing force before losses

F_u - ultimate strength of given grade of steel

F_y - yield stress for given grade of steel

F_y^p - yield stress of prestressing steel

F_y^r - yield stress of longitudinal reinforcing steel

H - height of stud shear connector

I_s - moment of inertia of steel beam

I_{sc} - moment of inertia of composite steel section

I_{ss} - moment of inertia of steel beam with coverplates

I_T - moment of inertia of transformed composite beam section

M_{DL} - bending moment due to dead load

M_{LL} - bending moment due to live load plus impact

N - number of shear connections required by fatigue criteria

N_1 - number of shear connectors required between end support and point of maximum positive moment by static (ultimate strength) criteria

N_2 - number of shear connectors required between point of maximum positive moment and point of adjacent maximum negative moment by static criteria

P - lesser value of P_1 and P_2

P_1 - ultimate strength of steel beam in tension

P_2 - ultimate strength of concrete slab in compression

P_3 - ultimate strength of longitudinal reinforcing steel in tension

P_4 - ultimate strength of prestressing steel in tension

Q - statical moment about neutral axis of composite section, of the transformed compressive concrete area for positive moment and for negative moment when the slab is prestressed, or the area of reinforcement embedded in the concrete for negative moment when the slab is not prestressed.

S_r - range of horizontal shear per linear inch at the junction of the slab and girder at point in span under consideration

S_s - section modulus of steel beam

S_u - ultimate strength of shear connector

T - tensile stress

V_r - range of shear due to live loads plus impact

Z_r - allowable range of horizontal shear on an individual shear connector

b - effective width of concrete slab

c - thickness of concrete slab

d - diameter of stud shear connector

d_s - depth of steel beam

e - eccentricity of prestressing steel above neutral axis of transformed composite section

f'_c - ultimate compressive strength of concrete

f_r - range of stress in top flange of composite steel section due to live load plus impact

n - ratio of modulus of elasticity of steel to that of concrete

p - percentage of longitudinal reinforcing steel in concrete slab

s - center to center spacing of shear connectors

w - unit weight of concrete

y_b - distance from neutral axis to bottom of composite beam section

y_{bs} - distance from neutral axis to bottom of steel beam with coverplates

y_t - distance from neutral axis to top of composite beam section

y_{ts} - distance from neutral axis to top of steel beam with coverplates

α - fatigue coefficient for stud shear connectors

ϕ - reduction factor

σ_{all} - allowable compressive stress for steel section

σ_{bc} - stress in extreme bottom fiber of transformed composite beam section

σ_{bs} - stress in extreme bottom fiber of composite steel section

σ_{ps} - stress due to application of prestressing force

σ_{tc} - stress in extreme top fiber of transformed composite beam section

σ_{tr} - stress in top layer of longitudinal reinforcing steel

σ_{ts} - stress in extreme top fiber of steel section

TABLES AND FIGURES

TABLE 1

MECHANICAL PROPERTIES OF STEEL

Type of Specimen	No. of Tests	Yield Point (ksi) Mean	Static Yield Stress (ksi) Mean	Tensile Strength (ksi) Mean
Web W21x62	3	37.5	34.4	60.8
Flange W21x62	4	34.4	32.0	58.8

TABLE 2

RESULTS OF CONCRETE CYLINDER TESTS

	A S T M				Cured With Slab	
Test Beam	No. of Tests	f'_c (psi)	No. of Tests	E_c (ksi)	f'_c (psi)	E_c (ksi)
PSC-1S	8	4760	2	3665	4810	3235
PSC-2S (Slab)	10	4330	4	3380	4370	
PSC-2S (Grout)					7070	

TABLE 3

ROLLED STEEL BEAM PROPERTIES

BEAM	AREA (in^2)	DEPTH (in)	FLANGE		WEB THICKNESS (in)	MOMENT OF INERTIA (in^4)
			WIDTH (in)	THICKNESS (in)		
PSC-1S	17.431	21.063	8.230	0.570	0.404	1251.465
PSC-2S	17.602	21.063	8.293	0.575	0.405	1267.489
*W21x62	18.300	20.990	8.240	0.615	0.400	1330.000

* From 1969 AISC Manual of Steel Construction

TABLE 4

PROPERTIES OF THE TEST BEAMS

BEAM	CROSS-SECTIONAL SLAB AREA (in^2)	LOCATION OF N.A. Y_b (in)	MOMENT OF INERTIA TRANSFORMED SECTION (in^4)	MOMENT OF INERTIA STEEL SECTION* (in^4)	COMPUTED ULTIMATE MOMENT (kip-in)
PSC-1S	366.54	20.303	3702	1638	7072
PSC-2S	365.83	20.042	3663	1654	7127

* Including STRESSTEEL bars

TABLE 5

STRESS RESULTANTS ON DESIGN SECTION

Analysis Section	DEAD LOAD		LIVE LOAD + IMPACT				Range of Shear V_r*
			Max. Pos. Effect		Max. Neg. Effect		
	V*	M**	V	M	V	M	
1	22.8	0	23.2	0	- 6.3	0	29.5
2	5.6	285	12.7	464	-11.9	-109	24.6
3	-11.6	226	5.0	539	-21.8	-218	26.8
4	-28.9	-180	1.0	333	-32.1	327	33.1
5	-46.2/ 51.8	-930	-1.0/ 34.7	73	-37.0/ - 2.6	-651	36.0/ 37.3
6	34.6	-67	25.5	254	- 4.6	-197	30.1
7	17.3	441	17.1	533	- 9.9	-145	27.0
8	0	624	10.0	638	-17.6	-145.0	27.6

* All shears and range of shear are in kips

** All moments are in kip-feet

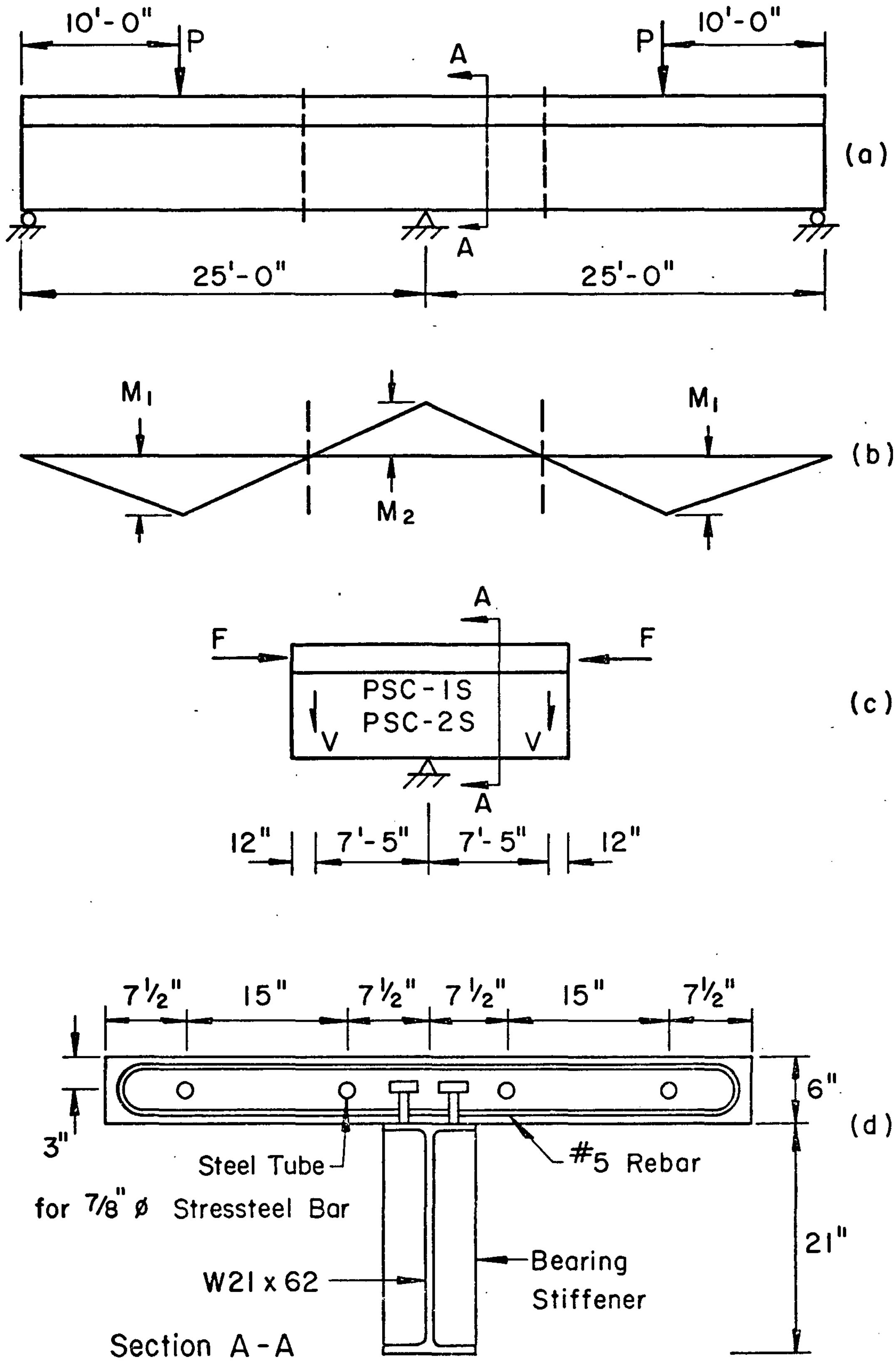


FIG. 1 TEST BEAMS PSC-1S AND PSC-2S

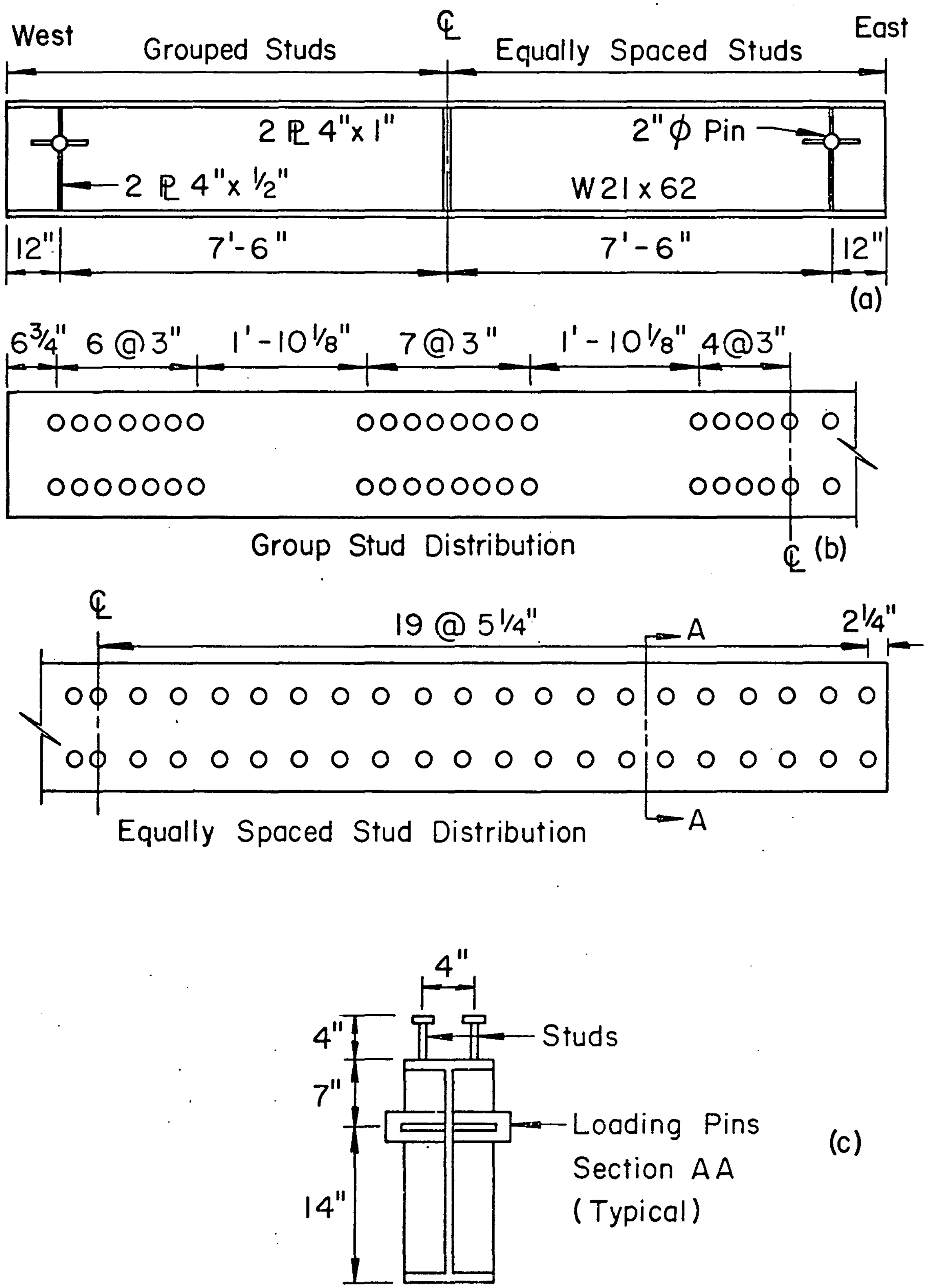


FIG. 2 FABRICATION DETAILS

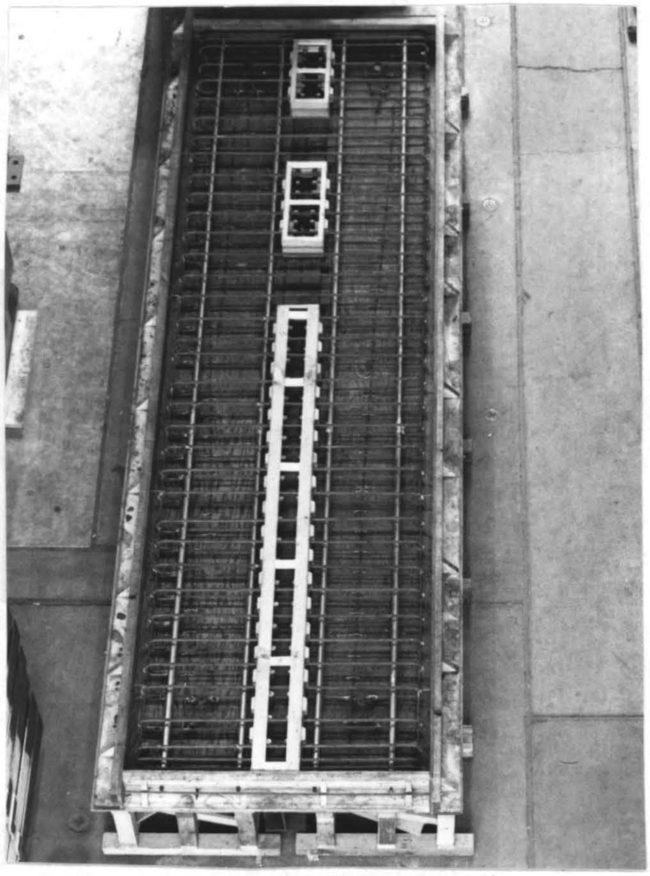


FIG. 3 FORMWORK FOR BEAM PSC-2S

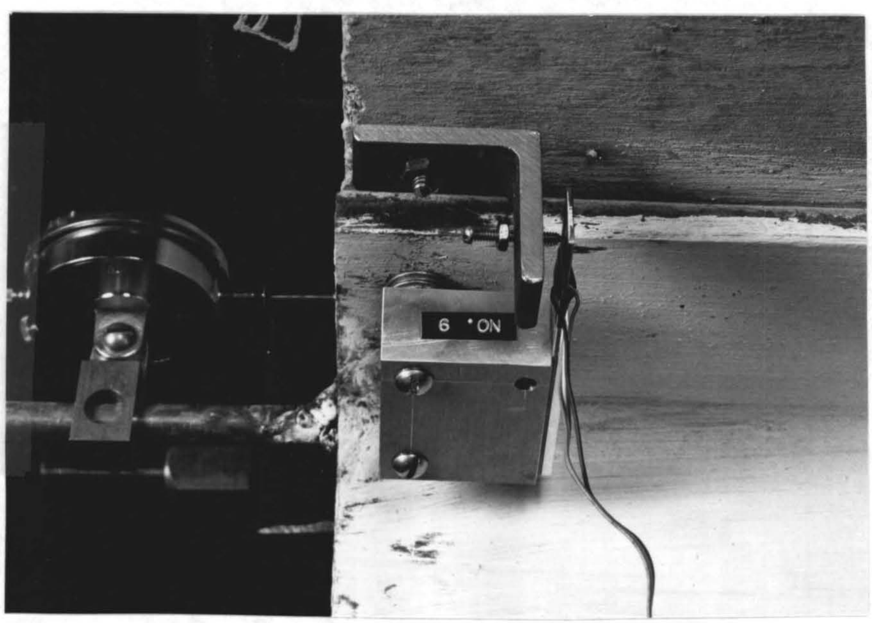


FIG. 4 ELECTRICAL SLIP GAGE

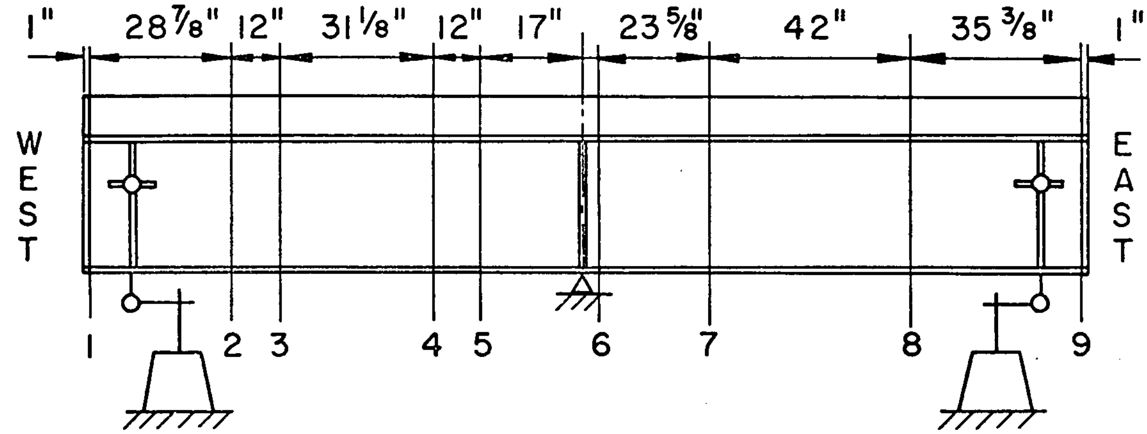


FIG. 5 LOCATION OF NINE INSTRUMENTED CROSS SECTIONS FOR BEAMS PSC-1S AND PSC-2S

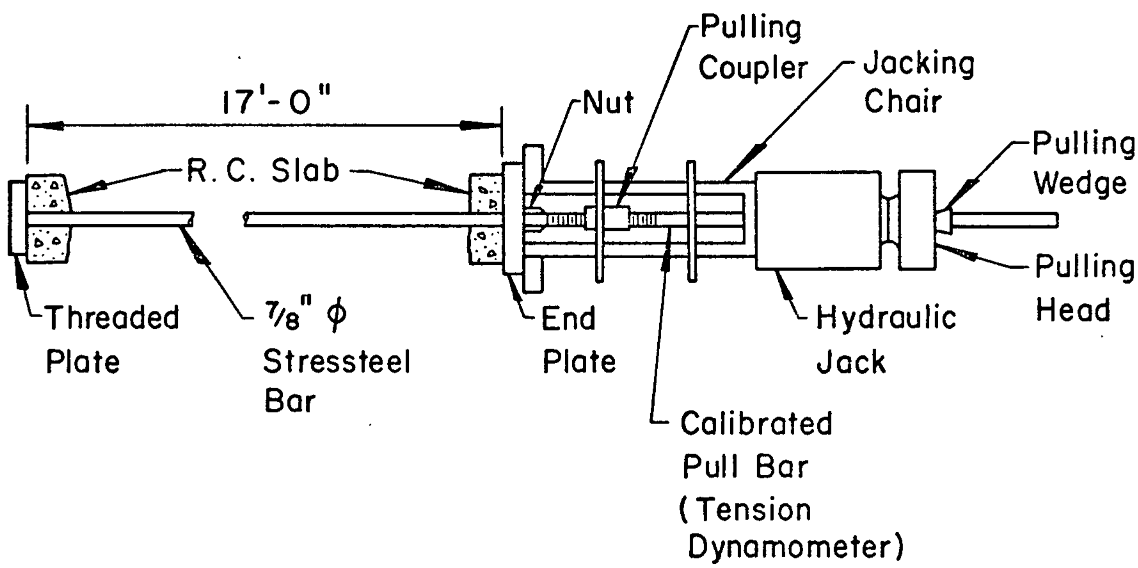


FIG. 6 PRESTRESSING SET-UP

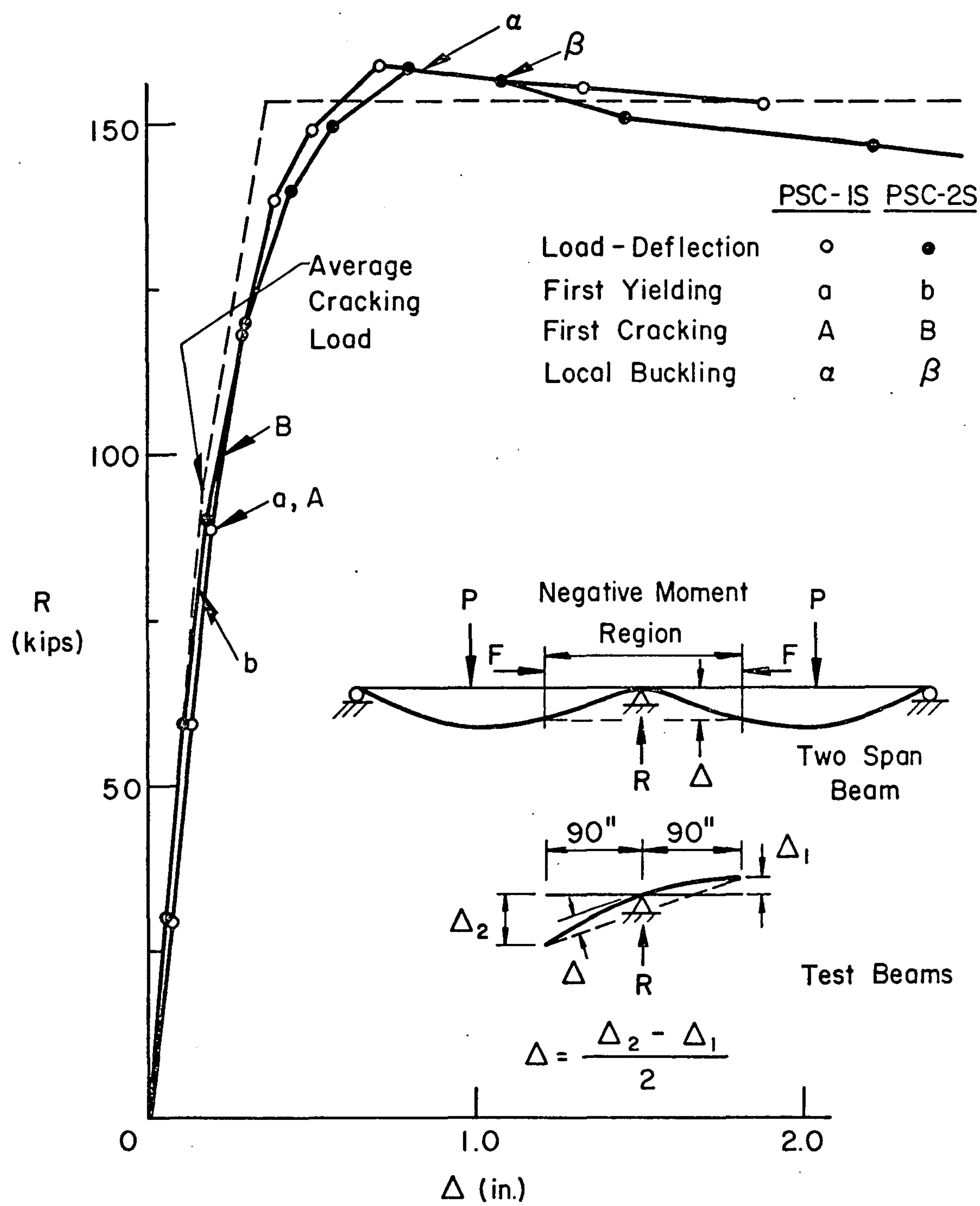


FIG. 7 LOAD-DEFLECTION BEHAVIOR OF BEAMS PSC-1S AND PSC-2S

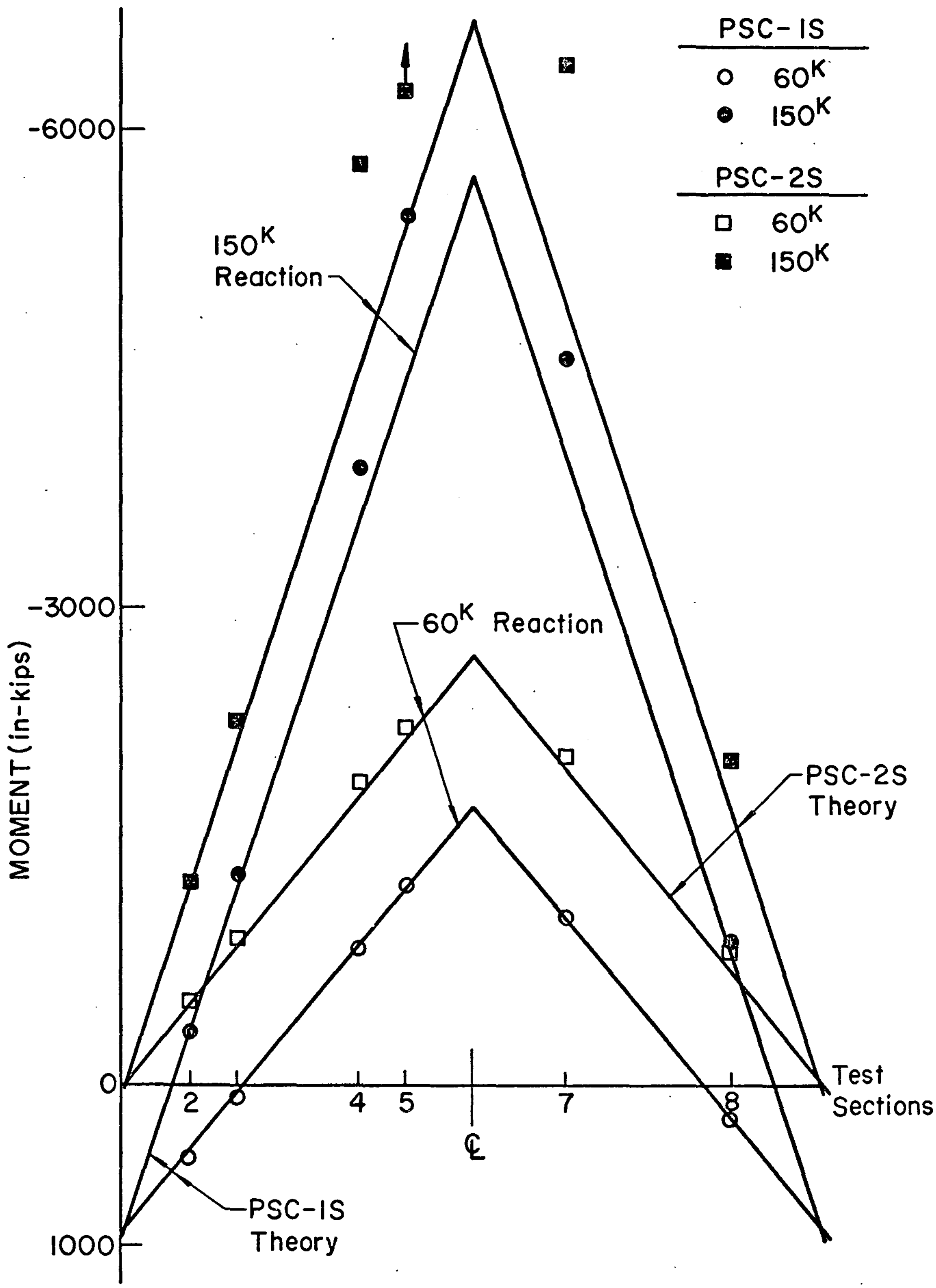


FIG. 8 BENDING MOMENT IN TEST BEAMS

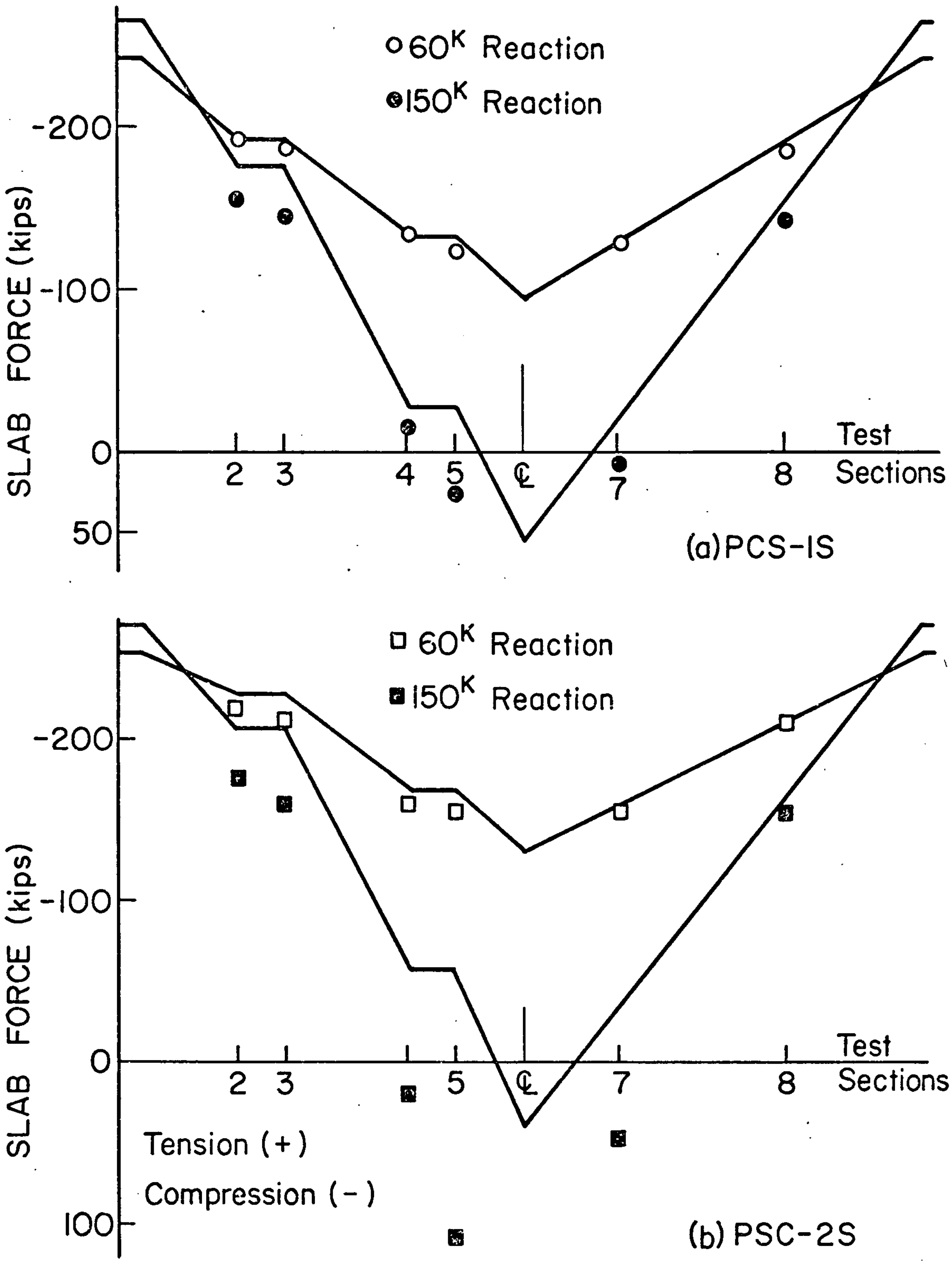


FIG. 9 SLAB FORCE IN TEST BEAMS

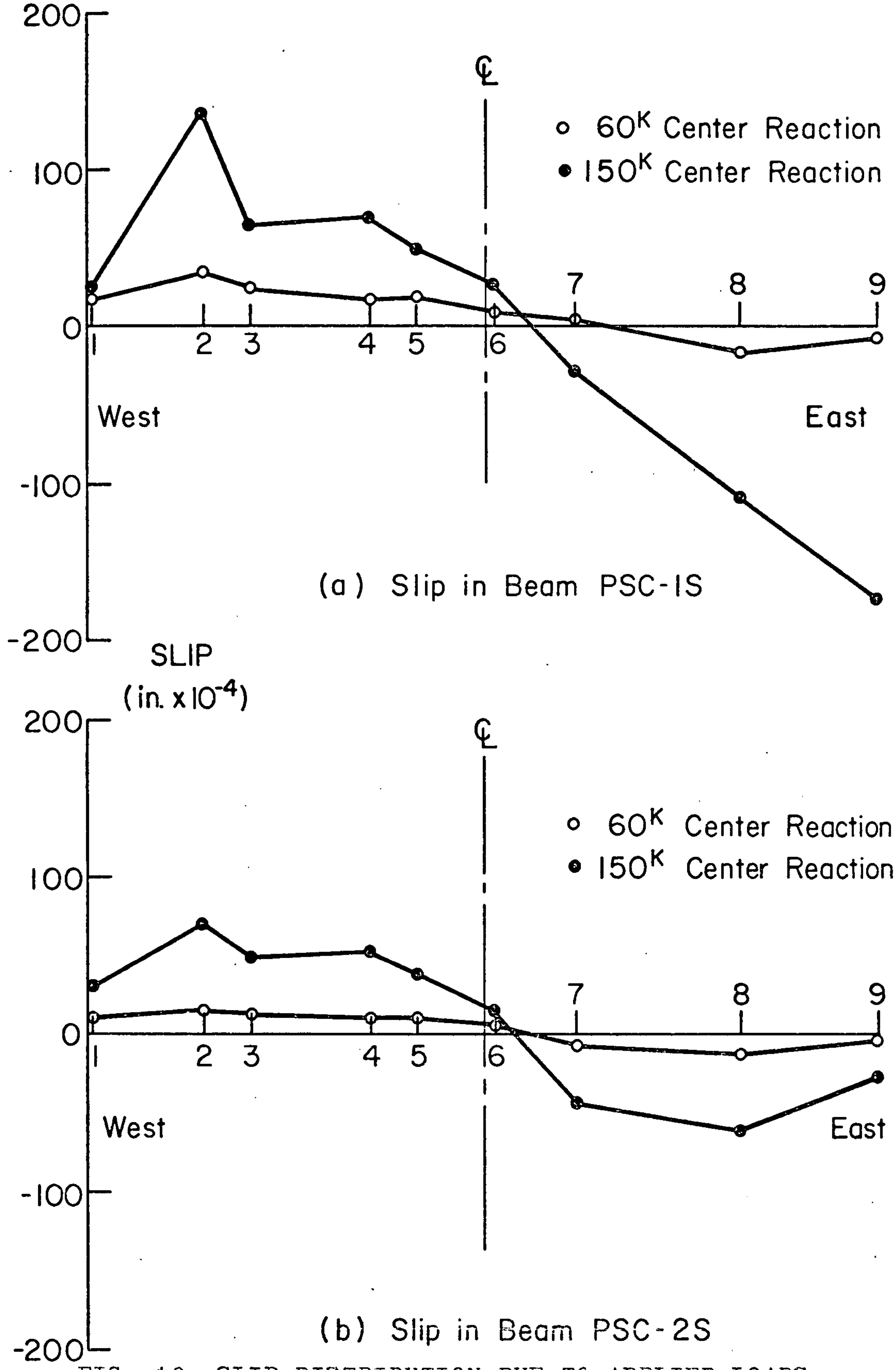


FIG. 10 SLIP DISTRIBUTION DUE TO APPLIED LOADS

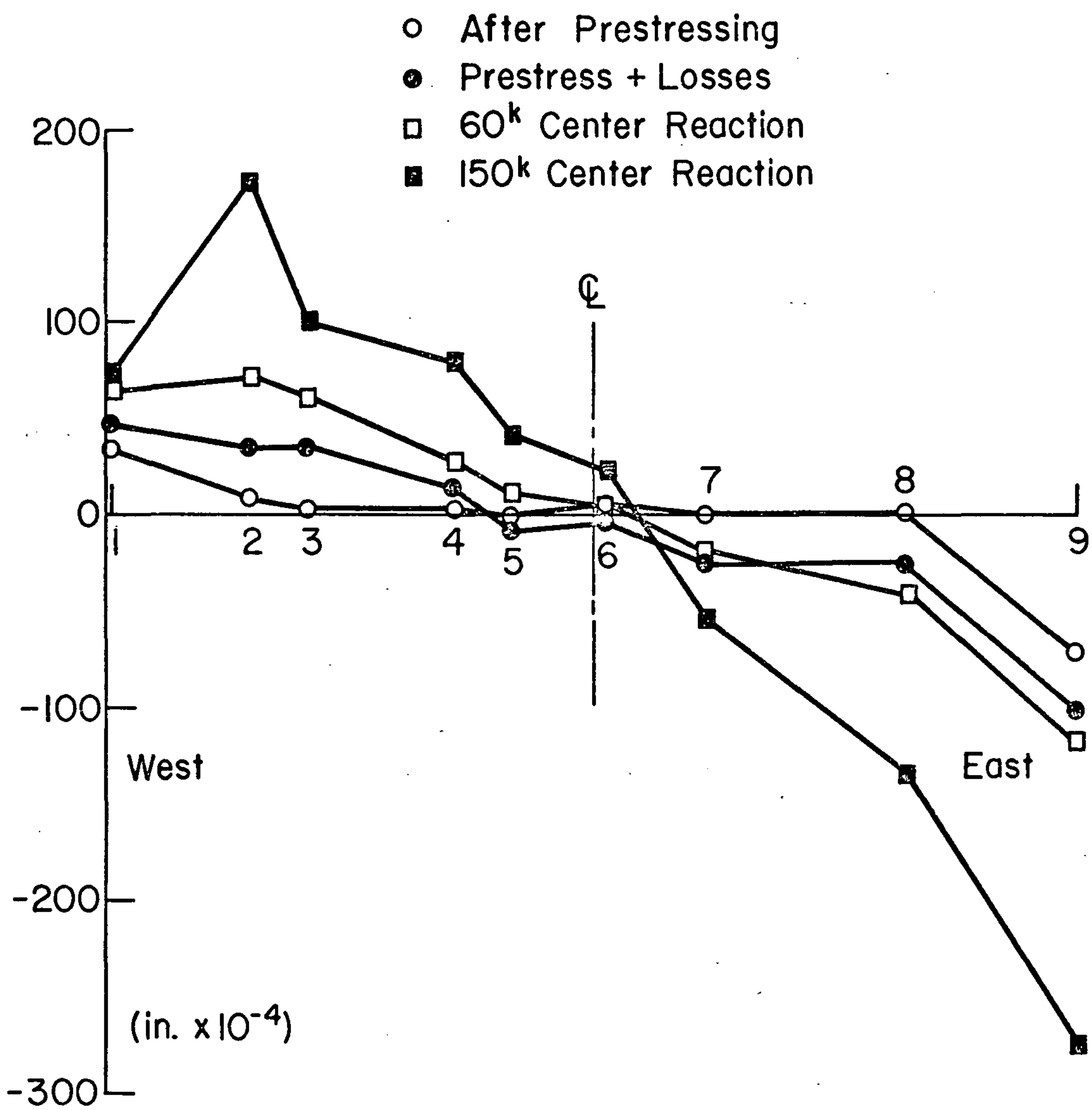


FIG. 11 TOTAL SLIP IN PSC-1S

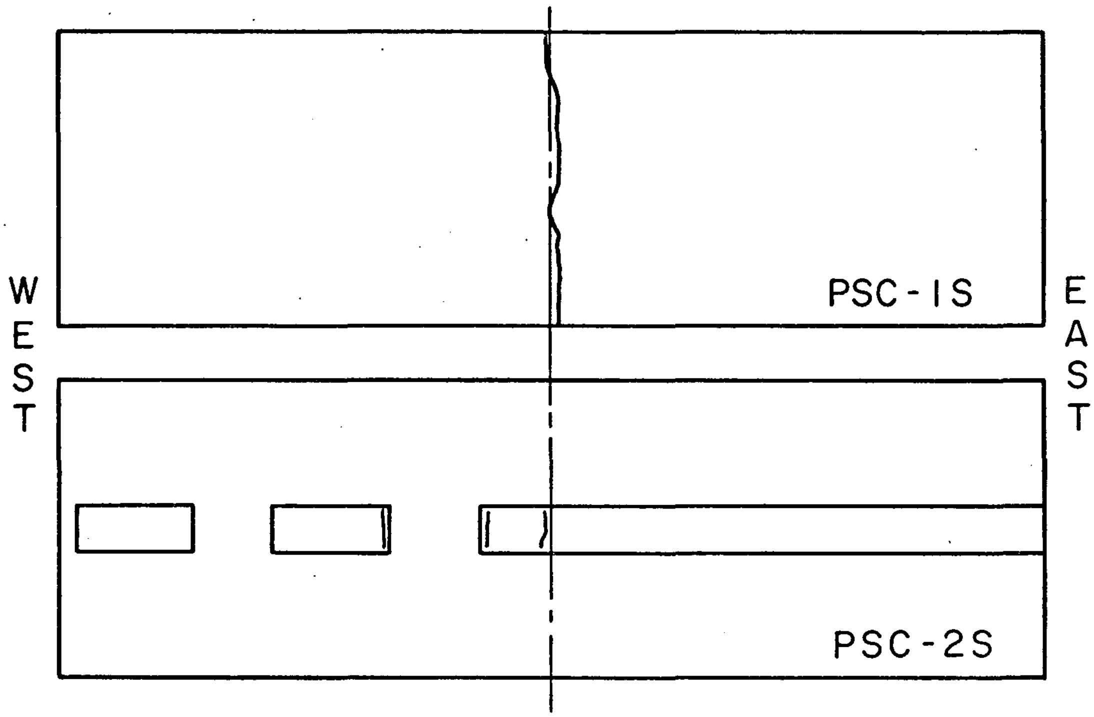


FIG. 12 SLAB CRACKING AT 90 KIPS

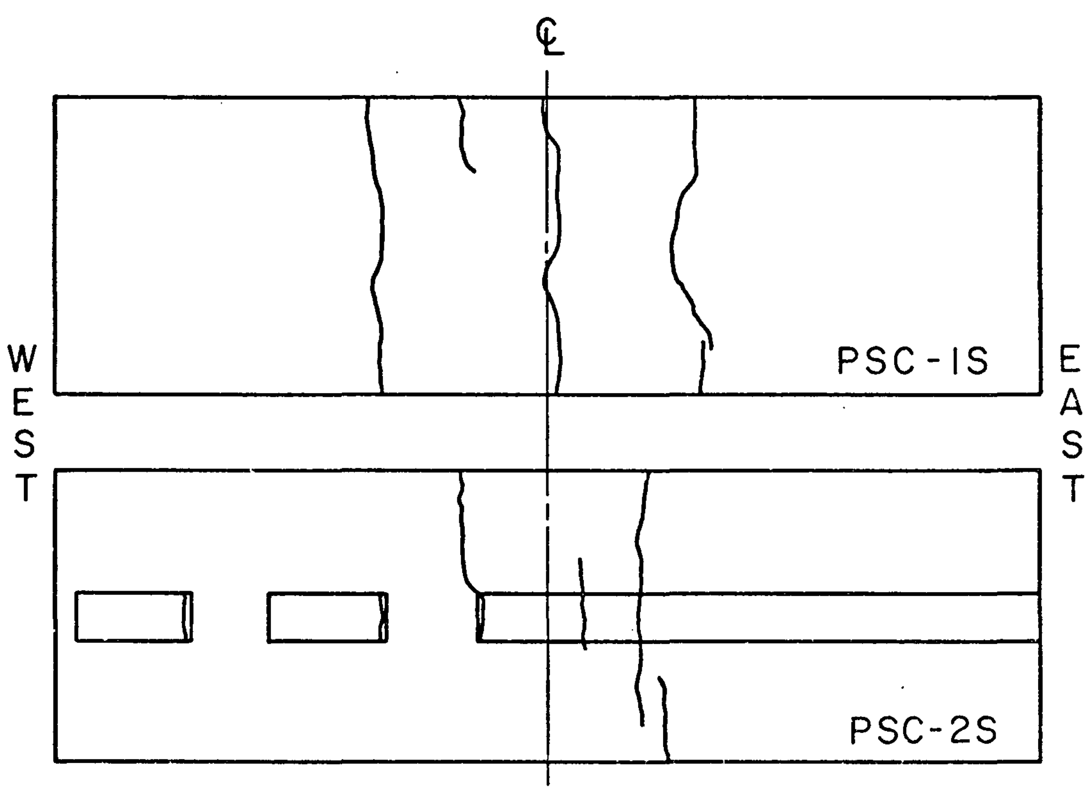


FIG. 13 SLAB CRACKING AT THE MAXIMUM LOAD

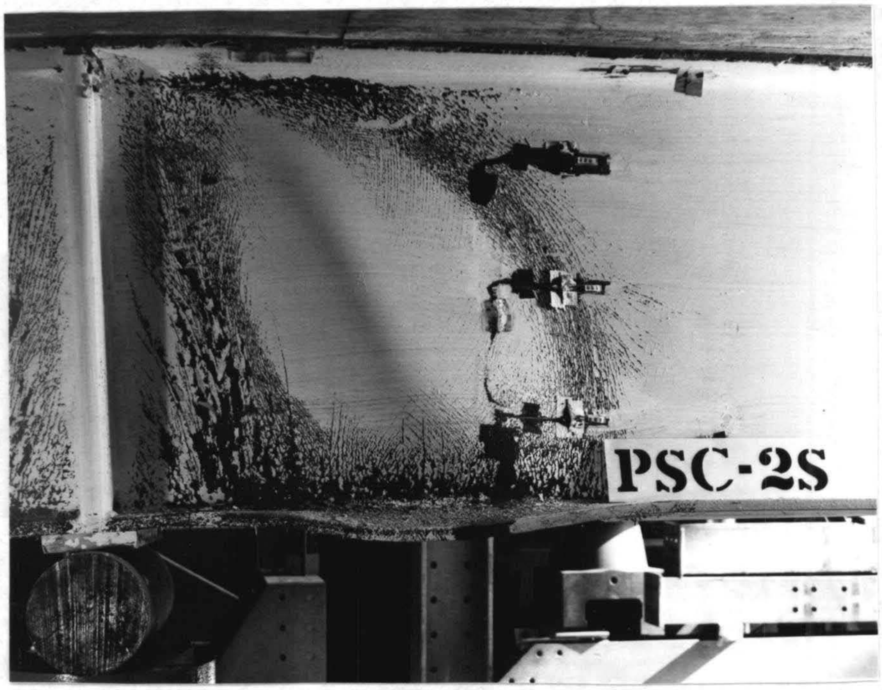


FIG. 14 LOCAL BUCKLING FAILURE MODE

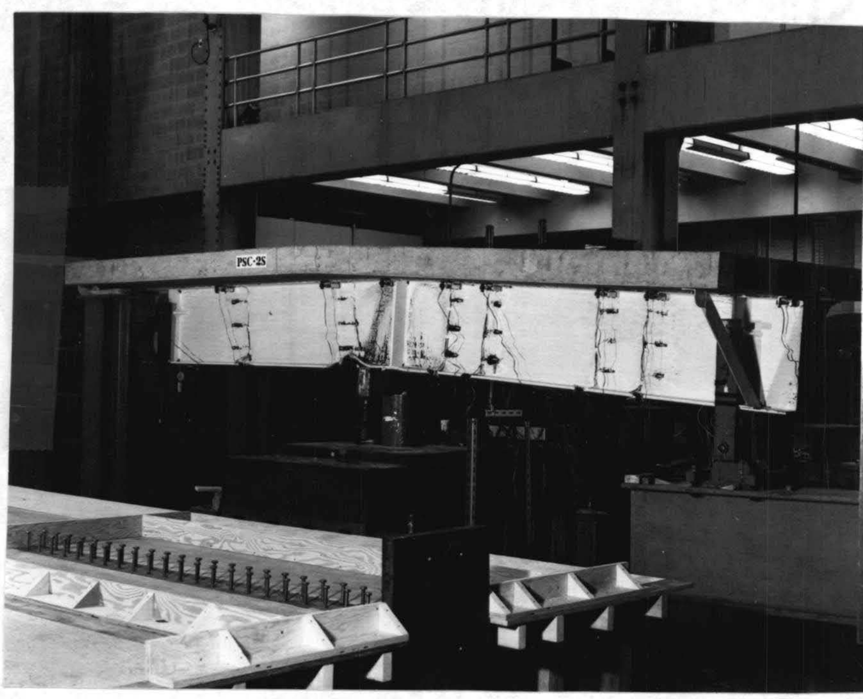


FIG. 15 TEST BEAM AFTER FAILURE

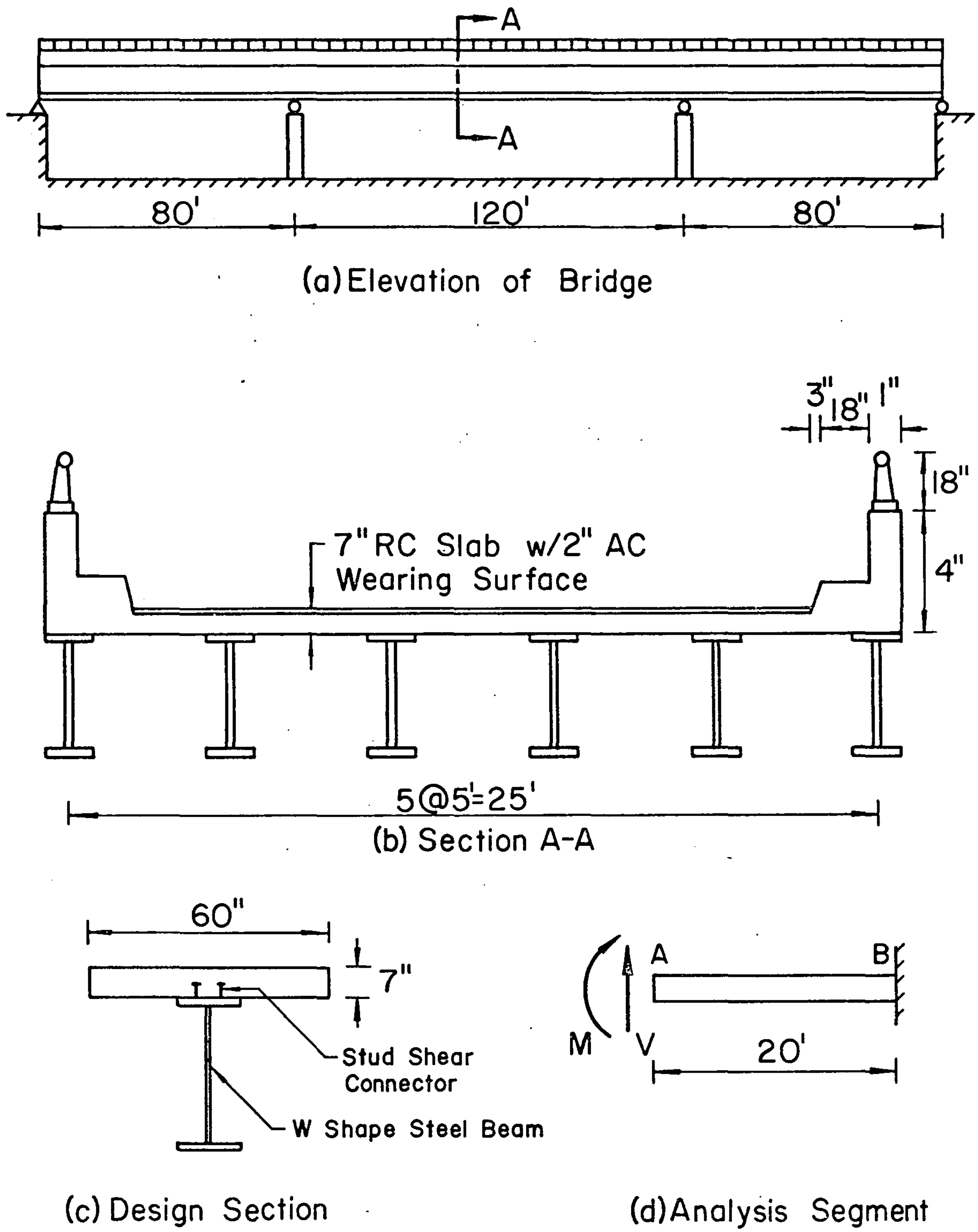


FIG. 16 BRIDGE USED FOR DESIGN EXAMPLES

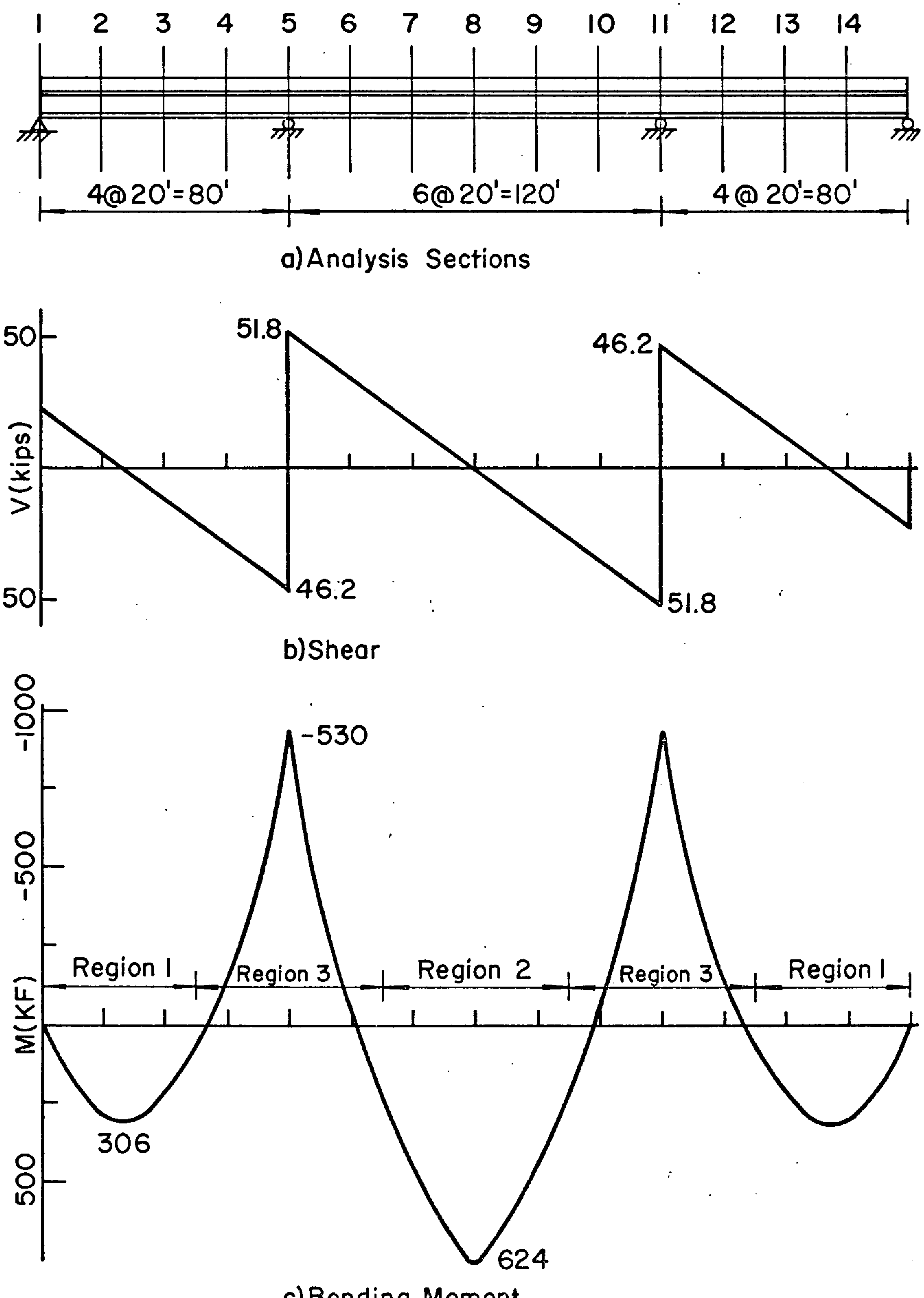


FIG. 17 STRESS RESULTANTS DUE TO DEAD LOAD

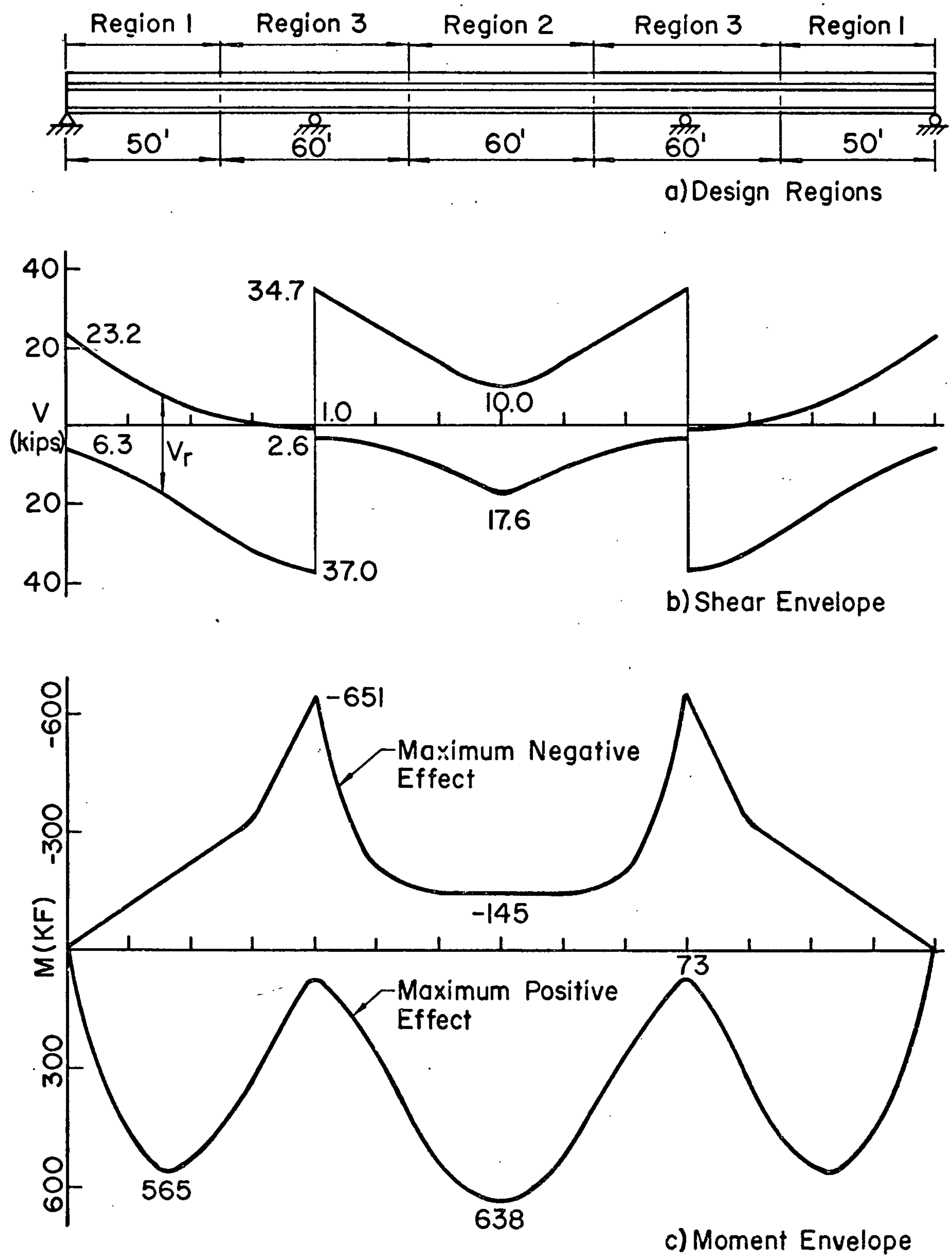


FIG. 18 STRESS RESULTANTS DUE TO LIVE LOAD PLUS IMPACT

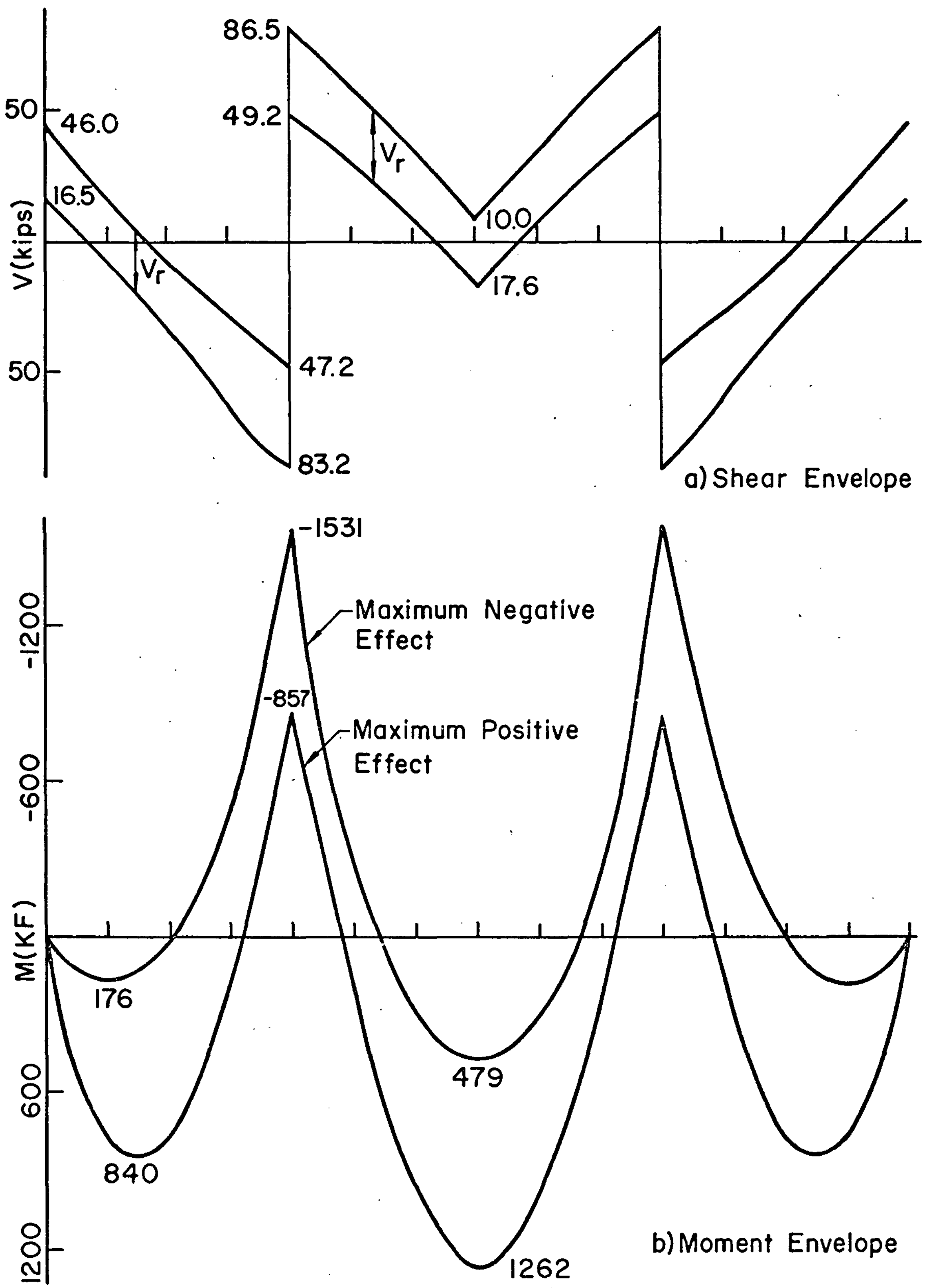


FIG. 19 STRESS RESULTANTS DUE TO ALL LOADS

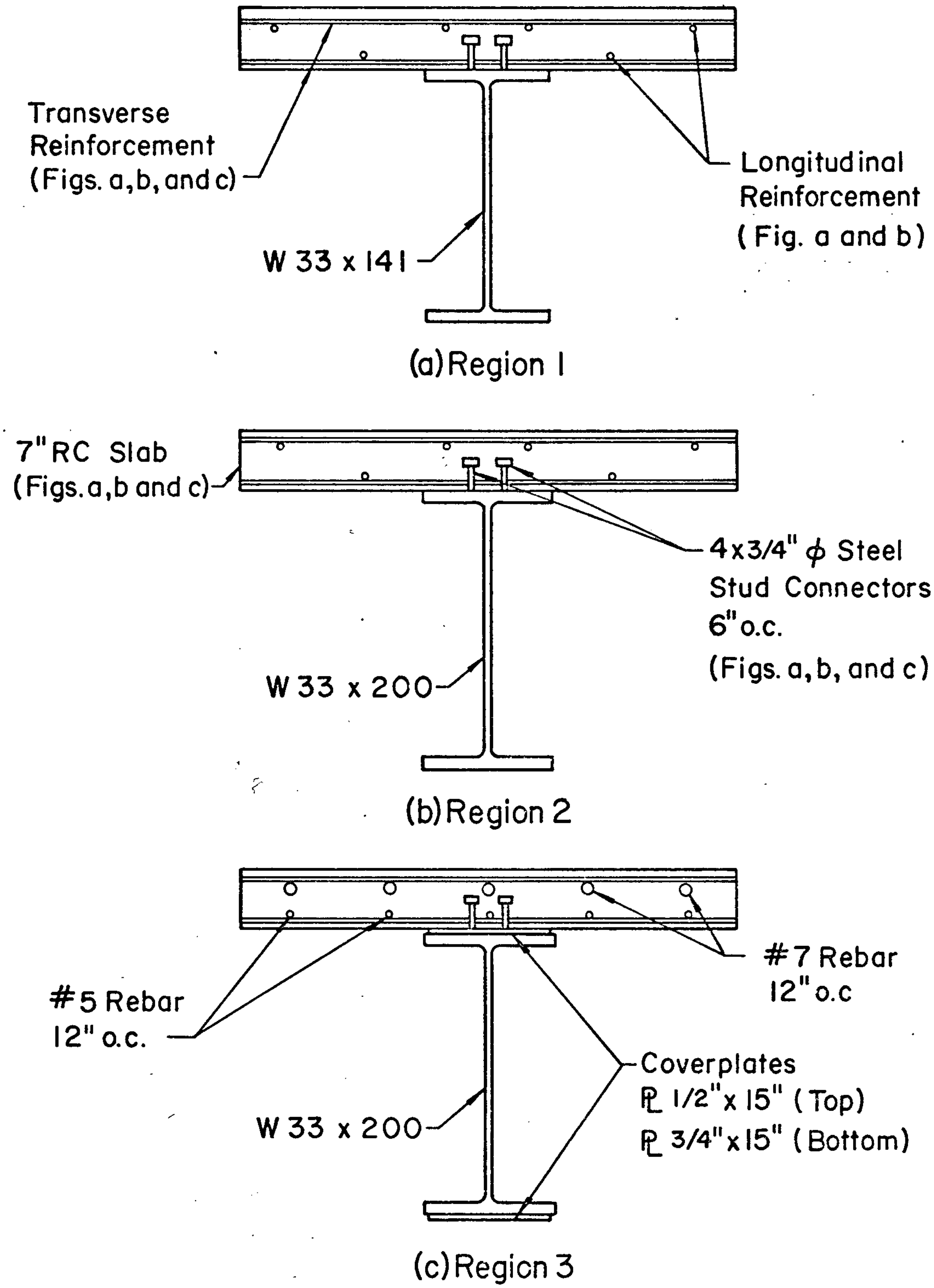
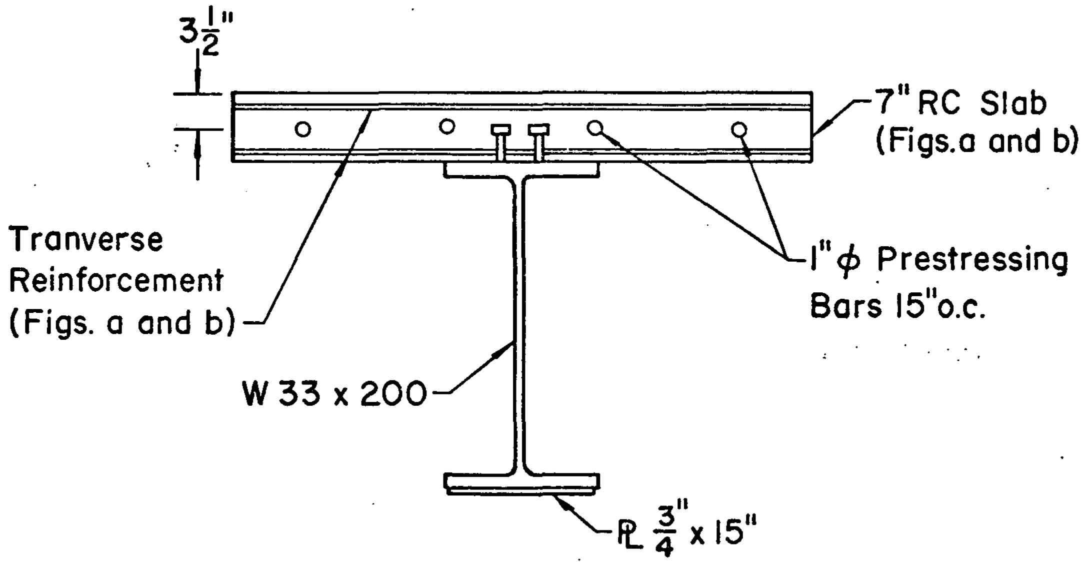
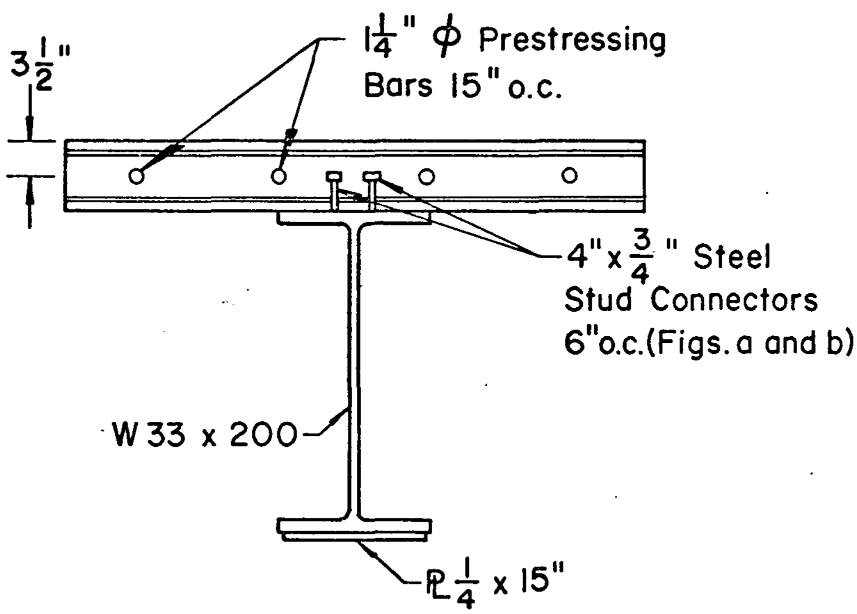


FIG. 20 SECTIONS FOR NONPRESTRESSED COMPOSITE BEAM

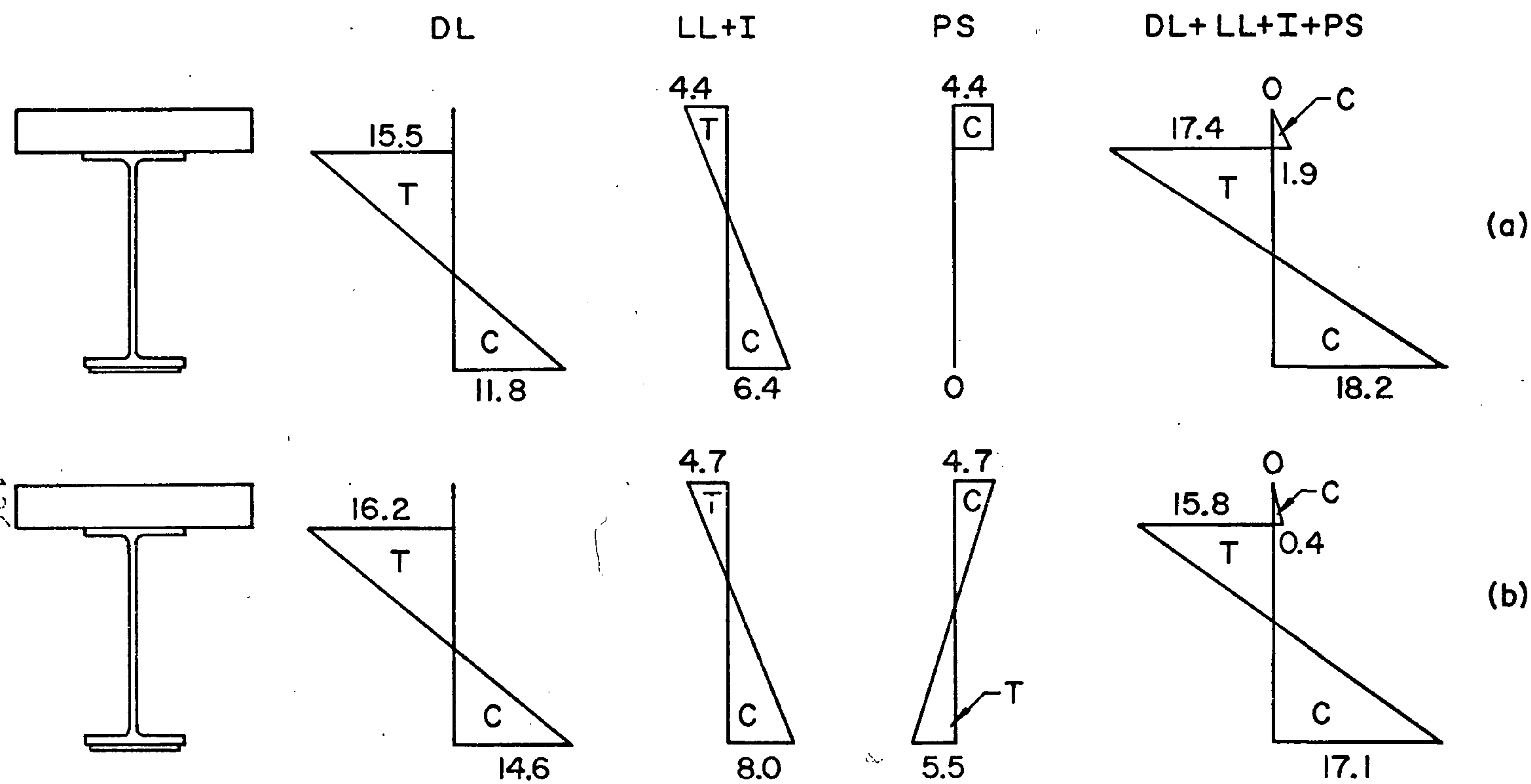


(a) Prestress Before Shear Connection



(b) Prestress After Shear Connection

FIG. 21 SECTIONS FOR PRESTRESSED COMPOSITE BEAMS
(REGION 3 ONLY)



Note: All stresses shown in ksi.

FIG. 22 STRESS DISTRIBUTION OVER INTERIOR SUPPORT OF PRESTRESSED COMPOSITE BEAMS

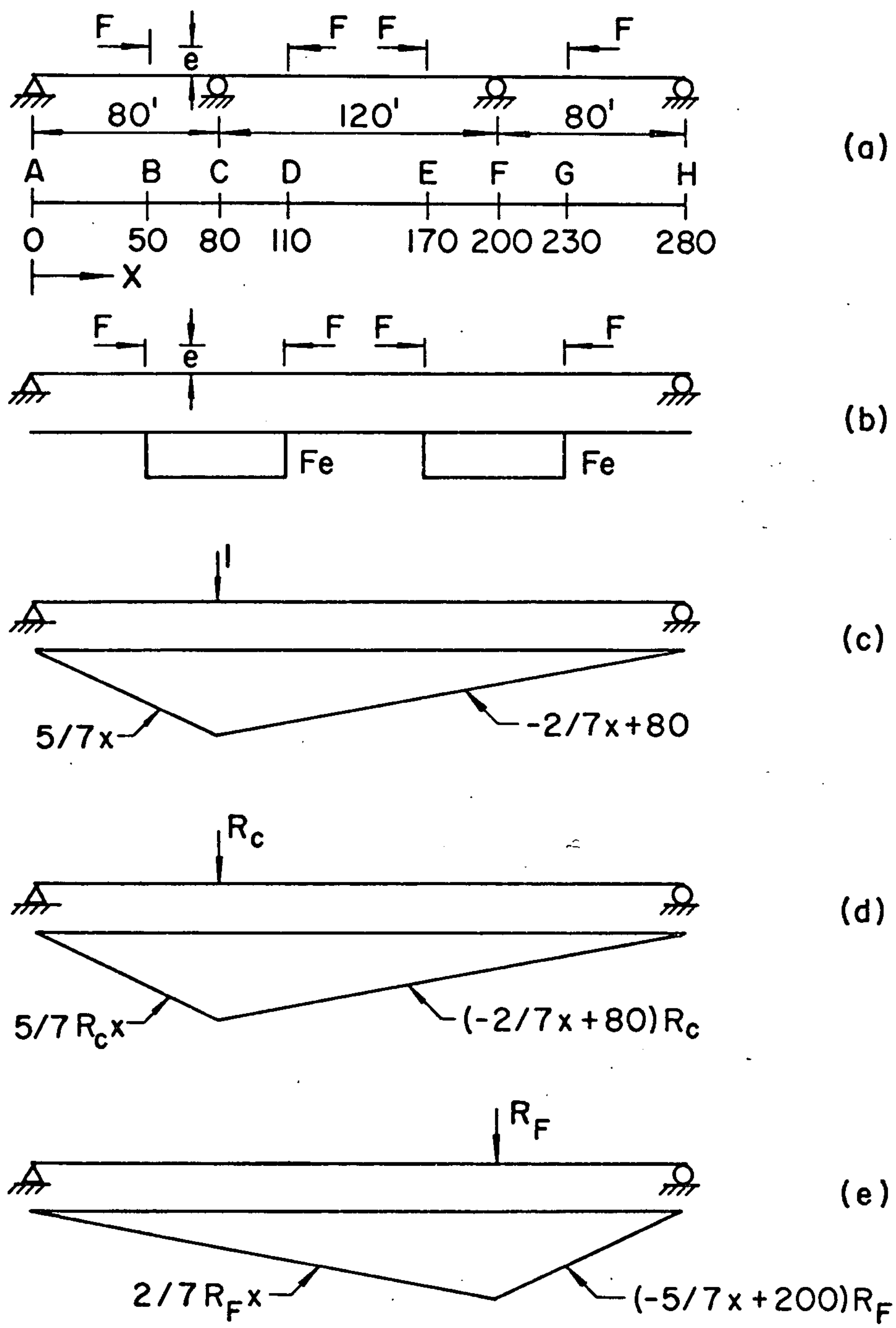


FIG. 23 MOMENT DIAGRAMS FOR VIRTUAL WORK ANALYSIS

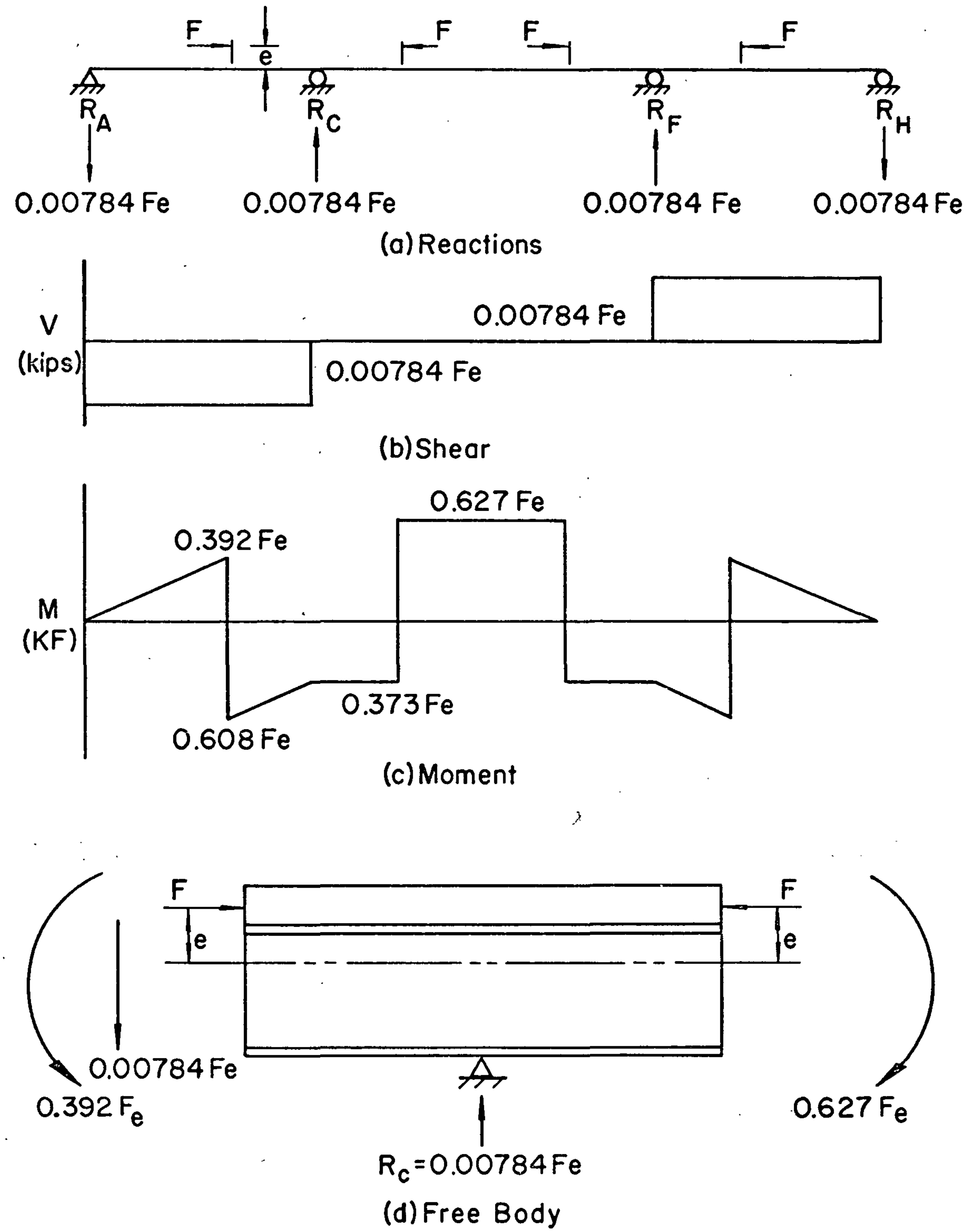


FIG. 24 EFFECTS OF APPLICATION OF PRESTRESSING FORCE AFTER SHEAR CONNECTION

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VITA

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